United States Department of Agriculture
Agricultural Research Service
Beltsville Agricultural Research Center-West
Beltsville, Maryland 20705

United States
Department of
Agriculture

Agricultural
Research
Service

ARS-14

January 1985

Open Channel Junctions With Supercritical Flow

Abstract

Rice, Charles E. 1985. Open Channel Junctions With Supercritical Flow, U.S. Department of Agriculture, Agricultural Research Service ARS-14, 34 p., illus.

The publication contains the results of preliminary tests using small-scale models to obtain information for the design of channel junctions for supercritical flows in rectangular junctions. A combination of many variables were tested, the most important of which were the upstream channel to lateral channel flow combinations, the upstream main channel to downstream channel width ratio, and the intersection angle between the main and lateral channels. Tables and illustrations showing the various combinations of variables studied are included. The findings provide direction for future model studies with supercritical flows in open channel junctions.

Keywords: flows, Froude number, jump, momentum principle, open channel junctions, supercritical flow, wave disturbance

The research reported in this publication was done in cooperation with the Oklahoma Agricultural Experiment Station.

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Open Channel Junctions With Supercritical Flow

by Charles E. Rice[1]

Introduction

Lined channels on steep slopes are included in projects of the U.S. Department of Agriculture, Soil Conservation Service in the Western States of the United States, and in other States for limited reaches in urban areas. The velocities of water flow in these channels are supercritical (sometimes called rapid or shooting flow). At junctions of main channels with lateral channels, where local inflows enter the main channel, a hydraulic jump may form in one or both channels. This jump will increase the flow depth significantly above normal and require a large increase in the sidewall height in the vicinity of the junction. Or, the main channel and lateral flows may pass through the junction at velocities greater than critical and cause standing waves which have a height in excess of normal sidewall freeboard. These waves may oscillate back and forth across the channel width for a considerable distance downstream before being damped by frictional forces. Higher than normal sidewalls may be required for a distance downstream as well as in the vicinity of the junction. General design information for junctions of this type is not available, and the consensus among hydraulicians is that such information cannot be adequately developed by either theoretical or empirical methods. The momentum principle can provide some insight into the flow behavior, but does not provide sufficient information for design of the junctions. Studies of physical models are required to determine the flow conditions and sidewall height requirements.

This publication presents the results of preliminary physical model tests designed to develop generalized design criteria for combining supercritical flows in rectangular open channel junctions. Unfortunately, program redirection necessitated the termination of these tests prior to their planned completion. However, these preliminary test results provide insight into open channel junction design with supercritical flows and identify the pertinent variables that affect the flow behavior in the junction area. Also, the results can provide guidance and direction for future model tests with supercritical flows in open channel junctions.

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Review of Literature

One of the first publications reporting on flow in open channel junctions was by Taylor (1944). His results are for combining subcritical flow only. Using the principle that the net force acting on a fluid is equal to the rate at which the sum of the pressure and momentum of the system is changed (momentum principle), he developed equations to define the behavior of the system. The conformity of the actual behavior to theory is discussed, and an empirical equation is included which can be applied to rectangular channels. There was good agreement between theory and experiment for 45° junctions, and a lack of agreement for 135° junctions. Taylor concluded that "generalization of the results was not possible, or even desirable, so no attempt has been made to present a mathematical statement applicable to every type of stream intersection."

Webber and Greated (1966) also investigated the flow behavior at the junction of rectangular channels with subcritical flows. They obtained theoretical flow patterns by the method of conformal mapping and used these results to develop a hydraulically efficient boundary profile. A theoretical solution for the flow pattern agreed reasonably well with an experimental solution.

Greated (1968) studied supercritical flow in rectangular open channels with 60° junctions. For horizontal channels with supercritical flow at relatively high Froude numbers (6 to 11) in all three channels, he was able to predict the wave positions by direct application of the momentum principle. Friction forces were ignored, and the waves were assumed to extend an infinite distance downstream. He made no attempt to calculate the wave heights, suggesting that a much more thorough investigation would be required, as the shape of the junction boundaries would be an important factor.

Gildea and Wong (1967) present design information on concrete-lined open channels and show how model tests results can modify and improve proposed designs. They stated: "When two high-velocity flood channels are joined in a confluence, it is necessary that the differential in elevation between the water surface of each stream be held to a minimum. The angle between the centerlines of the two adjoining channels. . . should be small (not exceeding 12 degrees) and if possible, the angle should be zero. . . If the angle is zero, the width of the channel at the confluence will be equal to the sum of the widths of the two channels plus the thickness of the dividing wall between the two channels."

Wong and Robles (1971) present information obtained from the design of flood-control improvements on southern California coastal streams. They wrote:

> Because of the many variables such as the angle of intersection, shape and size of channels, rate and type of flow, each junction has to be treated differently. . . Flow characteristics for these major junctions were analyzed by the momentum principle and verified by model tests. The experimental results for many junctions substantiated those calculated theoretically by the momentum equations, which are expressed in the following simplified forms:
>
> 1. For a rectangular main channel of constant width
>
> $$\frac{Q_3^2}{gA_3} + \frac{By_3^2}{2} = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2}(\cos \Theta) + \frac{By_1^2}{2} \quad \ldots\ldots(1)$$
>
> 2. For a rectangular main channel of unequal width upstream and downstream of the junction
>
> $$\frac{Q_3^2}{gA_3} + \frac{B_3y_3^2}{2} = \frac{Q_1^2}{gA_1} + \frac{Q_2^2}{gA_2}(\cos \Theta) + \frac{B_3y_1^2}{2} \quad \ldots.(2)$$
>
> in which. . . Q = discharge in ft^3/s (m^3/s); B = width of channel in ft(m); A = cross-sectional area of water prism in ft^2 (m^2); y = depth of flow in ft (m); . . . Θ = angle of intersection of channel centerlines in degrees; and g = gravitational constant in ft/s/s (m/s/s).[2]

The subscripts 1, 2, and 3 refer to the upstream main channel, the lateral channel, and the downstream channel, respectively. Strictly, each term of equations (1) and (2) should by multiplied by γ, the unit weight of water, to express the terms in force units. However, omission of γ for simplicity does not change the equality.

Wong and Robles (1971) also reported: "The results from a number of model studies indicated good flow characteristics with very little wave formation and turbulence at the junction if the following criteria are used as design guides: (1) Design water-surface elevations in the two adjoining channels should be nearly equal at the upstream end of the confluence; (2) the angle of channel intersection should be preferably zero but not greater than 12°; (3) favorable flow conditions can be achieved with proper expansion in width of main channel downstream of the junction; and (4) flow depth at the junction should not exceed 85% to 90% of critical depth (Froude number greater than 1.20 to maintain stable rapid flow through the junction)."

Behlke and Pritchett (1966) did a general study of supercritical flow in open channels with intersection angles up to 45° at various Froude numbers from about one to about seven. Methods were presented for the determination of the height of wall pileup which occurs under conditions of design flow and under unbalanced conditions of flow in only one of the channels. A design was recommended which minimized the wave problem in laboratory experiments. This design differs from a simple channel junction in that the main channel is widened at the junction to accommodate the increased discharge of the side channel, and a baffle, which is a sloping extension of the main channel, is extended downstream in front of the side channel.

Bowers (1950) conducted specific model studies to aid in the design of open channel junctions for a storm water disposal system for the Naval Auxiliary Station, Whiting Field, Fla. Some of the lateral channels entered the main channel at a 90° angle. Dependent on junction design, the discharges, velocities, and related phenomena of flow in the vicinity of the junction, a hydraulic jump may form in one or both of the inlet channels. If, however, the flow passes through the junction at velocities greater than the critical, standing waves with a height in excess of the normal freeboard may form and continue to oscillate back and forth across the channel for a considerable distance downstream. Pressure-momentum relationships were utilized in an attempt to analyze the behavior of the junction. Bowers comments, ". . . while the results were of considerable interest. . . they gave only a partial solution to the problem."

[2]The detailed derivation of these equations is presented in appendix VI of the Department of the Army (1970).

Froude Number Considerations

The Froude number has an important indirect use in describing the flow in the three channels. The Froude number, $F = V/(gD)^{0.5}$, can be defined as the ratio of the channel velocity V to the velocity of a gravity wave $(gD)^{0.5}$. Here D is the flow depth, and g is the gravitational constant. Since waves are caused by geometric discontinuities and/or flow disturbances at a junction, that definition has significance to this study.

Froude Number Equal to One

The Froude number also describes the type of flow. A Froude number of one indicates that the flow is at the velocity and depth of minimum energy content. In many cases flow will pass through critical depth downstream of the channel junction. This is a control point for beginning computations for the downstream water surface profile which determines the downstream sidewall height. A Froude number of one also indicates that the actual and wave velocities are equal.

Froude Number Near One

Flows close to critical depth ($F = 1$) should be avoided, if at all possible, because small changes in energy cause large changes in flow depth (Chow 1959), standing surface waves that may overtop the channel sidewall, and unstable flow conditions at junctions. Channel slope adjustments to avoid the range of Froude numbers between 0.8 and 1.2 are recommended by knowledgeable hydraulicians.

Froude Numbers Less Than One

A Froude number less than one indicates that the velocity is subcritical (tranquil flow). Equations 1 and 2 apply when the Froude number at the junction is less than one in all three channels.

Froude Numbers Greater Than One

A Froude number greater than one indicates that the velocity is supercritical (rapid flow). Rapid flow always occurs upstream of a hydraulic jump. If junction disturbances create sufficient tailwater, these disturbances may cause a hydraulic jump to form. The proximity of the jump to the junction is important in determining surface disturbances in the vicinity of the junction. Also, the height of the jump determines the sidewall height required to contain the flow. Equations to determine the jump height are well established and require only the Froude number and the depth of flow upstream of the jump. However, the exact location of the jump—and the distance upstream requiring higher than normal sidewalls—is difficult to determine.

For the limited tests reported herein the Froude numbers cover a very limited range—1.8 to 2.3—and, thus, are barely within the range of direct jumps which occur when the Froude number is greater than 1.7.

Flow Conditions

A general description of flow conditions in the junction area will be helpful in the discussion of the objectives and results of this preliminary investigation.

No Lateral Flow

With no flow from the lateral and supercritical flow in the upstream main, the main flow will expand into the lateral and impinge on the downstream sidewall of either the lateral or main. The wave thus created will oscillate back and forth across the downstream channel until it is gradually dissipated by friction. Higher than normal sidewalls may be required for some distance downstream of the junction to contain the crests of these waves.

No Upstream Main Flow

With supercritical flow in the lateral and no flow in the upstream main, the lateral flow will impinge on the main channel sidewall opposite the junction. The wave thus created will oscillate across the downstream main channel and may require higher than normal sidewalls for some distance downstream of the junction.

Flows in Both Lateral and Upstream Main

With certain combinations of supercritical flow in the lateral and upstream main, flow conditions in the downstream main will be improved. A flow from the lateral can prevent the upstream main flow from entering the lateral, prevent wave formation from impinging on the lateral or main sidewall, and eliminate the oscillating waves and improve flow conditions in the downstream channel. Similarly, the flow from the upstream main may prevent the lateral flow from impinging on the main sidewall opposite the junction and eliminate the wave from this source. Some combinations of flow will improve and others will worsen the flow in the downstream main.

Research Objectives

The junction can be made more efficient and simpler if it is designed so that the jump or wave disturbance caused by the intersecting flow will remain within the junction area and not move upstream into the main channel. If the jump moves upstream of the junction into the main channel, the maximum depth of flow in the junction and the location of the jump in the upstream main cannot be adequately predicted. Thus, the height of the sidewalls and the distance upstream that the increased height must extend will not be known. If the jump remains within the junction area, the depth of flow in the upstream main channel can be calculated as the normal depth, and no increase in the main channel sidewall height will be required upstream of the junction. However, it is probably not possible, for all flow conditions, to prevent a jump from forming at the lateral channel exit and moving upstream into the lateral channel.

The principal objective of this research, then, is to establish the geometric and hydraulic parameters and limits that will ensure that disturbances caused by the junction and combinations of flows from the lateral and upstream main remain in the immediate vicinity of the junction. A related objective is to develop criteria for predicting the maximum height of water surface in the junction area and the water surface profile in the downstream channel. This information will enable a designer to reliably predict the maximum sidewall heights required in the upstream main channel, lateral channel, and downstream main channel and to provide adequate, but not excessively high, sidewalls.

Procedure

Preliminary studies with small-scale models were used to obtain information for the design of channel junctions for supercritical flow in rectangular channels. The combinations of variables tested are listed in table 1: Q is the channel flow; Θ is the angle of intersection

Table 1.—Variable combinations tested[1]

Test No.	Q_1	Θ	S_m	S_l	B_1	B_2
2.2, 2.4-2.6	0.76	90	0.00465	0.00289	1.00	1.00
2.7-2.15	1.50	90	.00465	.00289	1.00	1.00
2.16-2.22	.40	90	.00465	.00289	1.00	1.00
2.23-2.27	.76	90	.00465	.00289	1.00	1.00*
2.28-2.32	.76	90	.00465	.00289	.67	1.00*
2.33-2.37	1.50	90	.00465	.00289	.67	1.00*
3.1-3.6,#3.61,#3.62	.81	90	.0198	.0105	.67	1.00
3.7-3.15	1.50	90	.0198	.0105	.67	1.00
4.1-4.7	.75	60	.0198	.00887	.67	1.01
4.8-4.15	1.50	60	.0198	.00887	.67	1.01
4.16-4.23	.40	60	.0198	.00887	.67	1.01
4.24-4.31	1.50	60	.0198	.00849	.67	1.00**
4.32-4.38	.75	60	.0198	.00849	.67	1.01**
4.39-4.46	0	60	.0198	.00849	.67	1.01**
4.47-4.52	.40	60	.0198	.00887	.67	.66
4.53-4.58	.65	60	.0198	.00887	.67	.66
4.59-4.63	.91	60	.0198	.00887	.67	.66
4.65-4.70	1.20	60	.0198	.00887	.67	.66
4.71-4.76	1.50	60	.0198	.00887	.67	.66
5.1-5.8	1.50	60	.0198	.00887	.67	.66
5.9-5.16	.75	60	.0198	.00887	.67	.66
6.0-6.7	.40	60	.0198	.00887	.50	.66
6.8-6.15	.75	60	.0198	.00887	.50	.66
6.17-6.21	1.20	60	.0198	.00887	.50	.66
6.22-6.28	1.50	60	.0198	.00887	.50	.66
7.1-7.10,#7.50	.40	30	.0198	.0105	.50	.66
7.11-7.20,#7.49	.75	30	.0198	.0105	.50	.66
7.21-7.30	1.20	30	.0198	.0105	.50	.66
7.31-7.40	1.50	30	.0198	.0105	.50	.66
7.41-7.47	0	30	.0198	.0105	.50	.66
8.1-8.10,#8.50	.40	30	.0198	.0105	.67	.67
8.11-8.20	.75	30	.0198	.0105	.67	.67
8.21-8.30	1.20	30	.0198	.0105	.67	.67
8.31-8.40	1.50	30	.0198	.0105	.67	.67
8.41-8.47	0	30	.0198	.0105	.67	.67
9.1-9.10,#9.50	.40	30	.0198	.0105	.67	.98
9.11-9.20	.75	30	.0198	.0105	.67	.98
9.21-9.30	1.20	30	.0198	.0105	.67	.98
9.31-9.40	1.50	30	.0198	.0105	.67	.98
9.41-9.47	0	30	.0198	.0105	.67	.98

[1] Q_1 is upstream main channel discharge in cubic feet per second;
Θ is intersection angle between main and lateral channels in degrees;
S_m is bottom slope of main channel in feet per foot;
S_l is bottom slope of lateral channel in feet per foot;
B_1 is upstream main channel width in feet;
B_2 is lateral channel width in feet;
B_3 is downstream channel width in feet, equal to 1.0 ft for all tests.
*Lateral channel bottom raised 0.125 ft above main channel.
**Lateral channel bottom raised 0.143 ft above main channel.

between the main and lateral channels; S_m and S_l are the bottom slopes of the main and lateral channels, respectively; and B is the channel width. The subscripts 1, 2, and 3 refer to the upstream main channel, lateral channel, and main channel downstream, respectively.

The model channel system was fabricated and installed in an indoor laboratory equipped with a recirculating water supply. The channel walls and bottom were constructed of aluminum alloy; however, clear acrylic plastic was used for the sidewalls in the junction vicinity to permit visual observation of the flow behavior. The channel slopes were adjustable to permit variation of the flow depths and velocities. Figure 1 is a line drawing showing the water supply, channel and measurement system. Figure 2 is a definition sketch of the junction. Photographs of the system appear in figure 3.

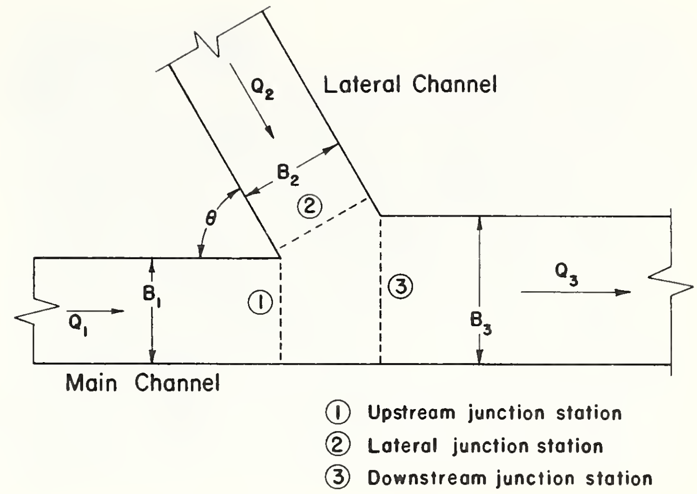


Figure 2.—Definition sketch of junction and vicinity.

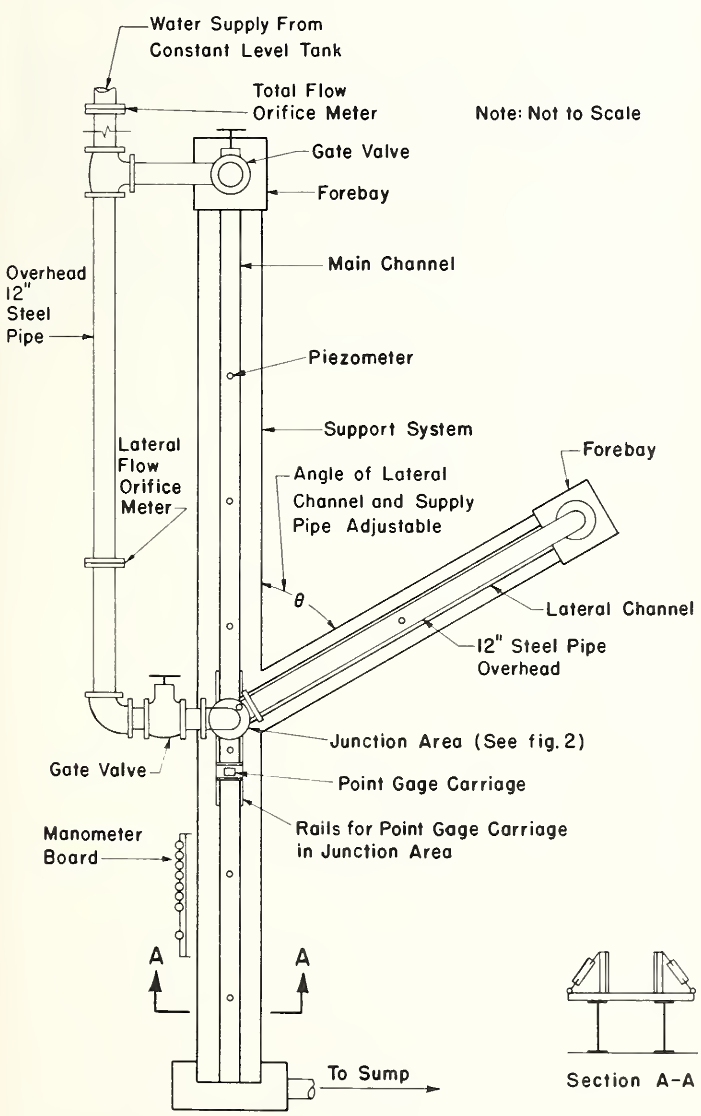
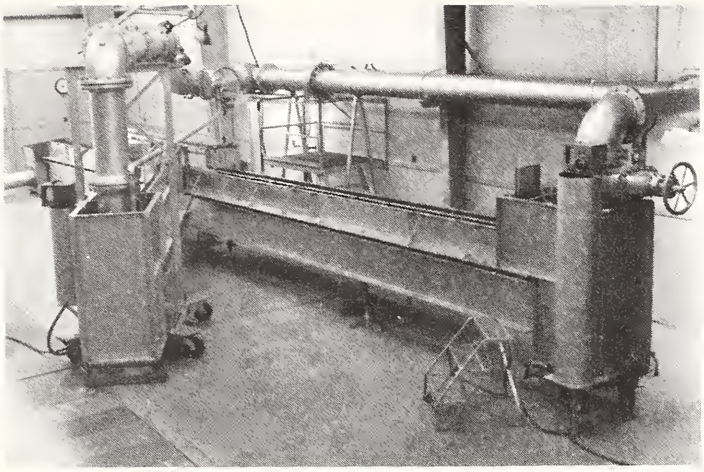


Figure 1.—Plan view drawing of channel system.

For each test, appropriate channel geometric variables were set, the channels' slopes, widths, and bottom elevations were measured; the gages were zeroed; the test flows were set; and flow conditions in the system were allowed to reach equilibrium. Measurements were then taken to determine the flow rates, water surface elevations, and water temperatures. The testing procedure in general was: set an upstream main channel discharge; take appropriate readings and observations; introduce a small lateral channel discharge and repeat readings and observations; increase the lateral channel discharges until the desired data have been collected. Then set a new upstream main channel discharge and repeat the procedure.

Flows were measured with orifice meters and air-water differential manometers. Piezometers were located on 6-ft centers along the main and lateral channels (fig. 1) and connected to 2-in gage wells surmounted with point gages. A point gage on a traveling carriage was used to measure the water surface elevations in the junction vicinity when the water surface was not turbulent. Because the water surface in the junction vicinity was very turbulent at times, an electronic system was used to obtain point average and maximum water surface elevations in the junction vicinity for some of the tests. With this system a sine wave generator sent a signal through an upstream probe. When the upstream probe came in contact with the water surface, a downstream probe (set about 0.75 inches downstream of the upstream probe and at a lower elevation to be in continuous contact with the water) received the signal and



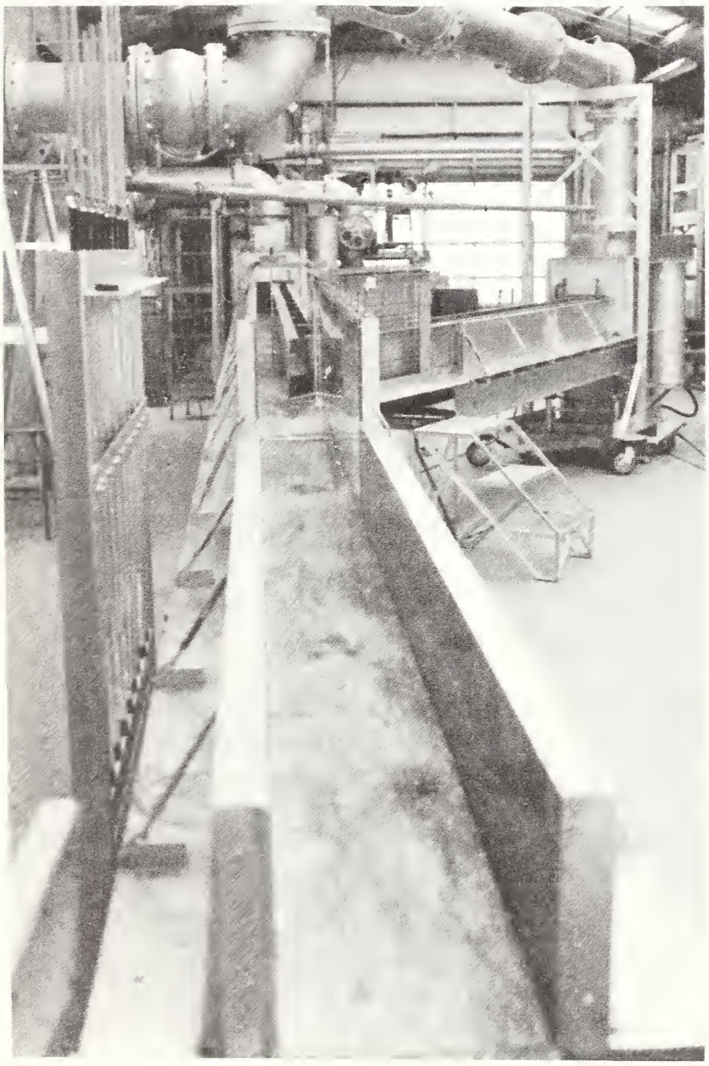


Figure 3.—General view of channel system for test series 7, $B_1/B_3 = 0.50$, $B_1/B_2 = 0.66$, $\Theta = 30°$, $B_3 = 1.01$ ft: *A*, view from upstream; *B*, view from downstream.

transmitted it to a pulse counter. The ratio of the recorded count to the generated frequency was a measure of the percentage of time the upstream probe was in contact with the water. Readings taken with the probe at three or more elevations were related to the percentage of time the probe was in contact with the water. These data permitted calculation of the approximate average (50 percent time) and maximum (100 percent time) water surface elevations. A standard mercury thermometer was used to measure the water temperature. The measurements were supplemented with photographs and the observer's comments.

The data were analyzed, and information was developed that provides guidance for the design of open channel junctions with supercritical flows. The momentum principle was applied to evaluate its ability to predict flow depths in open channel junctions with supercritical flows.

Before the experimental program began, tests were run to determine the Manning coefficient *n* for the test channel. In the range of flows used, the Manning coefficient *n* varied from about 0.00838 to 0.00885 with most values within the range 0.00855 to 0.00870. The variation of the Manning coefficient *n* with the flow rate was of a random nature. A constant value of 0.0086 was used in calculating normal depths and in other calculations requiring the Manning coefficient.

Test Data

The variables measured during the tests are the upstream main channel flow rate (Q_1), the lateral channel flow rate (Q_2), the water temperature, the flow depths at the upstream (D_1) and downstream (D_3) junction stations of the main channel, the flow depth in the lateral channel (D_2) at the junction entrance, and the maximum observed in the junction vicinity (D_m). These observed mean depths (weighted average point depths—specific force of mean depth equal to summation of specific forces of individual segments of cross section) with associated variables are presented in tables 2 through 9. Typical water surface profiles are presented in figures 4 and 5. In these figures, D_n is the normal depth and D_c is the critical depth in the upstream (D_{n1}, D_{c1}) and downstream (D_{n3}, D_{c3}) channels. Average water surface depths across the channel width in the junction vicinity and the downstream channel are presented in figures 6 and 7 for selected tests.

Table 2.—Series 2 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
2.02	0.762	0.229	0	0.232	0.762	0.230	0.230	54
2.04	.756	.434	.198	.445	.954	.310	.310	54
2.05	.765	.471	.268	.467	1.033	.349	.349	55
2.06	.750	.508	.351	.516	1.101	.387	.387	56
2.07	1.490	.344	0	.357	1.490	.360	.360	60
2.08	1.498	.562	.124	.575	1.622	.449	.449	60
2.09	1.506	.617	.226	.633	1.732	.521	.521	60
2.10	1.578	.674	.299	.685	1.877	.547	.547	58
2.11	1.490	.718	.436	.726	1.926	.568	.568	59
2.12	1.486	.774	.593	.789	2.079	.576	.576	59
2.13	1.501	.535	.045	.505	1.546	.448	.448	58
2.14	1.503	.553	.090	.554	1.593	.450	.450	59
2.15	1.502	.700	.372	.698	1.874	.576	.576	60
2.16	.400	.147	0	.153	.400	.142	.142	59
2.17	.402	.191	.015	.207	.417	.175	.175	59
2.18	.398	.211	.028	.224	.426	.184	.184	59
2.19	.399	.264	.060	.256	.459	.185	.185	59
2.20	.399	.286	.100	.288	.499	.207	.207	59
2.21	.407	.329	.160	.328	.567	.229	.229	62
2.22	.401	.367	.240	.368	.641	.261	.261	62
2.23	.757	.220	0	.093	.757	.221	.221	62
2.24	.758	.389	.125	.269	.883	.309	.309	62
2.25	.761	.449	.200	.312	.961	.350	.350	63
2.26	.759	.467	.269	.345	1.028	.375	.375	63
2.27	.735	.501	.339	.372	1.074	.385	.385	63
2.28	.756	.293	0	.053	.756	.179	.179	62
2.29	.763	.319	.124	.184	.887	.280	.280	62
2.30	.748	.372	.201	.252	.949	.288	.288	63
2.31	.761	.422	.281	.307	1.042	.360	.360	63
2.32	.757	.464	.332	.332	1.089	.397	.397	64
2.33	1.502	.466	0	.251	1.502	.308	.308	66
2.34	1.497	.481	.127	.275	1.624	.362	.362	68
2.35	1.500	.503	.223	.352	1.723	.422	.422	69
2.36	1.505	.626	.427	.498	1.932	.483	.483	69
2.37	1.515	.785	.800	.670	2.315	.622	.622	70

[1] Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

Table 3.—Series 3 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
3.01	0.812	0.210	0	0.097	0.812	0.137	0.227	72
3.02	.802	.208	.117	.207	.919	.187	.267	72
3.03	.807	.211	.176	.240	.983	.195	.320	74
3.04	.803	.211	.274	.320	1.077	.319	.479	74
3.05	.816	.215	.339	.357	1.155	.322	.483	75
3.06	0	.250	.357	.218	.357	.145	.315	79
3.61	.808	.228	.380	.389	1.188	.330	.503	76
3.62	.806	.230	.428	.425	1.234	.352	.532	77
3.07	1.499	.328	0	.154	1.499	.233	.356	77
3.08	1.494	.331	.124	.283	1.618	.278	.349	77
3.09	1.497	.332	.244	.354	1.741	.321	.421	78
3.10	1.483	.336	.439	.480	1.922	.481	.672	78
3.11	1.504	.756	.820	.708	2.324	.607	.835	79
3.12	0	.475	.931	.454	.931	.229	.573	79
3.13	1.537	.368	.535	.541	2.072	.502	.717	74
3.14	1.511	.648	.723	.648	2.234	.552	.770	75
3.15	1.525	.772	.926	.774	2.451	.581	.858	76

[1] Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

Table 4.—Series 4 test data[1]

Test No.	S_l	Q_1	D_1	B_2	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
4.01	0.00887	0.771	0.198	1.01	0	0.139	0.771	0.133	0.202	74
4.02	.00887	.749	.389	1.01	.442	.397	1.191	.358	.451	75
4.03	.00887	.748	.574	1.01	.751	.579	1.499	.423	.602	76
4.04	.00887	.737	.641	1.01	.938	.642	1.675	.436	.662	77
4.05	.00887	.752	.195	1.01	.199	.244	.951	.211	.307	77
4.06	.00887	.748	.196	1.01	.281	.295	1.029	.309	.439	78
4.07	.00887	.723	.190	1.01	.344	.332	1.067	.330	.467	78
4.08	.00887	1.497	.325	1.01	0	.131	1.497	.220	.343	79
4.09	.00887	1.489	.327	1.01	.126	.286	1.615	.278	.324	78
4.10	.00887	1.502	.332	1.01	.219	.344	1.721	.314	.361	79
4.11	.00887	1.498	.334	1.01	.349	.402	1.847	.367	.446	80
4.12	.00887	1.502	.416	1.01	.450	.460	1.952	.458	.628	79
4.13	.00887	1.519	.615	1.01	.718	.602	2.237	.579	.738	79
4.14	.00887	1.518	.634	1.01	.916	.714	2.434	.629	.786	79
4.15	.00887	0	.408	1.01	.925	.382	.925	.311	.504	79
4.16	.00887	.408	.297	1.01	.343	.321	.751	.248	.366	78
4.17	.00887	.411	.399	1.01	.498	.406	.909	.304	.432	78
4.18	.00887	.398	.523	1.01	.813	.517	1.211	.385	.538	78
4.19	.00887	.402	.115	1.01	0	.032	.402	.085	.127	77
4.20	.00887	.430	.121	1.01	.158	.183	.588	.183	.313	78
4.21	.00887	.402	.122	1.01	.247	.248	.649	.231	.346	78
4.22	.00887	.399	.119	1.01	.051	.118	.450	.115	.137	78
4.23	.00887	.399	.120	1.01	.101	.154	.500	.131	.227	78
4.24	.00849	1.499	.327	1.01	0	0	1.499	.227	.356	78
4.25	.00849	1.498	.340	1.01	.446	.314	1.944	.462	.627	78
4.26	.00849	1.520	.590	1.01	.705	.422	2.225	.559	.684	78
4.27	.00849	1.462	.676	1.01	.913	.547	2.375	.618	.777	80
4.28	.00849	0	.400	1.01	.930	.212	.930	.343	.626	80
4.29	.00849	1.506	.334	1.01	.336	.196	1.842	.348	.464	79
4.30	.00849	1.497	.328	1.01	.125	.125	1.622	.286	.331	79
4.31	.00849	1.494	.330	1.01	.218	.151	1.712	.319	.397	79
4.32	.00849	.754	.198	1.01	0	0	.754	.117	.199	62
4.33	.00849	.771	.206	1.01	.183	.069	.954	.241	.309	63
4.34	.00849	.764	.203	1.01	.265	.106	1.029	.338	.474	64
4.35	.00849	.725	.197	1.01	.339	.141	1.064	.356	.468	63
4.36	.00849	.763	.203	1.01	.447	.168	1.210	.355	.484	64
4.37	.00849	.754	.575	1.01	.749	.389	1.503	.440	.616	64
4.38	.00849	.766	.643	1.01	.931	.457	1.697	.484	.679	64
4.39	.00849	0	.203	1.01	.340	.094	.340	.168	.345	64
4.40	.00849	0	.269	1.01	.537	.141	.537	.231	.474	65
4.41	.00849	0	.323	1.01	.690	.175	.690	.283	.542	64
4.42	.00849	0	.395	1.01	.903	.208	.903	.343	.606	63
4.43	.00849	0	.454	1.01	1.112	.258	1.112	.389	.697	63
4.44	.00849	0	.523	1.01	1.420	.327	1.420	.412	.819	58
4.45	.00849	0	.562	1.01	1.596	.372	1.596	.509	.849	58
4.46	.00849	0	.635	1.01	1.905	.442	1.905	.588	.929	58
4.47	.00887	.398	.116	.66	0	.027	.398	.079	.115	56
4.48	.00887	.399	.117	.66	.050	.113	.449	.105	.125	56
4.49	.00887	.398	.123	.66	.100	.157	.498	.131	.226	56
4.50	.00887	.398	.122	.66	.200	.183	.598	.213	.314	56
4.51	.00887	.402	.191	.66	.280	.251	.682	.252	.347	56
4.52	.00887	.405	.241	.66	.302	.258	.707	.250	.361	56
4.53	.00887	.650	.176	.66	0	.036	.650	.110	.178	56
4.54	.00887	.652	.176	.66	.100	.183	.752	.173	.203	56
4.55	.00887	.651	.177	.66	.198	.219	.849	.202	.380	57
4.56	.00887	.657	.178	.66	.296	.281	.953	.294	.421	56
4.57	.00887	.657	.394	.66	.446	.378	1.103	.363	.452	52

Table 4. Continued

Test No.	S_l	Q_1	D_1	B_2	Q_2	D_2	Q_3	D_3	D_m	Tmp
4.58	.00887	.647	.501	.66	.606	.499	1.253	.390	.491	52
4.59	.00887	.910	.226	.66	0	.051	.910	.171	.242	56
4.60	.00887	.911	.226	.66	.199	.268	1.110	.236	.321	55
4.61	.00887	.909	.228	.66	.297	.313	1.206	.276	.475	55
4.62	.00887	.910	.405	.66	.492	.417	1.402	.383	.523	54
4.63	.00887	.910	.572	.66	.698	.532	1.608	.481	.631	52
4.65	.00887	1.197	.282	.66	0	.061	1.197	.182	.297	52
4.66	.00887	1.201	.289	.66	.199	.300	1.400	.273	.332	52
4.67	.00887	1.207	.293	.66	.298	.348	1.505	.302	.465	52
4.68	.00887	1.208	.369	.66	.499	.446	1.707	.414	.623	52
4.69	.00887	1.199	.578	.66	.704	.571	1.903	.509	.756	52
4.70	.00887	1.200	.706	.66	.906	.681	2.106	.606	.828	52
4.71	.00887	1.506	.330	.66	0	.069	1.506	.198	.343	52
4.72	.00887	1.504	.337	.66	.199	.337	1.703	.313	.377	52
4.73	.00887	1.505	.338	.66	.289	.383	1.794	.341	.429	52
4.74	.00887	1.505	.347	.66	.498	.467	2.003	.427	.718	52
4.75	.00887	1.509	.554	.66	.695	.573	2.204	.535	.775	52
4.76	.00887	1.506	.714	.66	.902	.666	2.408	.652	.918	53

[1]S_l is bottom slope of lateral channel in feet per foot;
Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
B_2 is lateral channel width in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

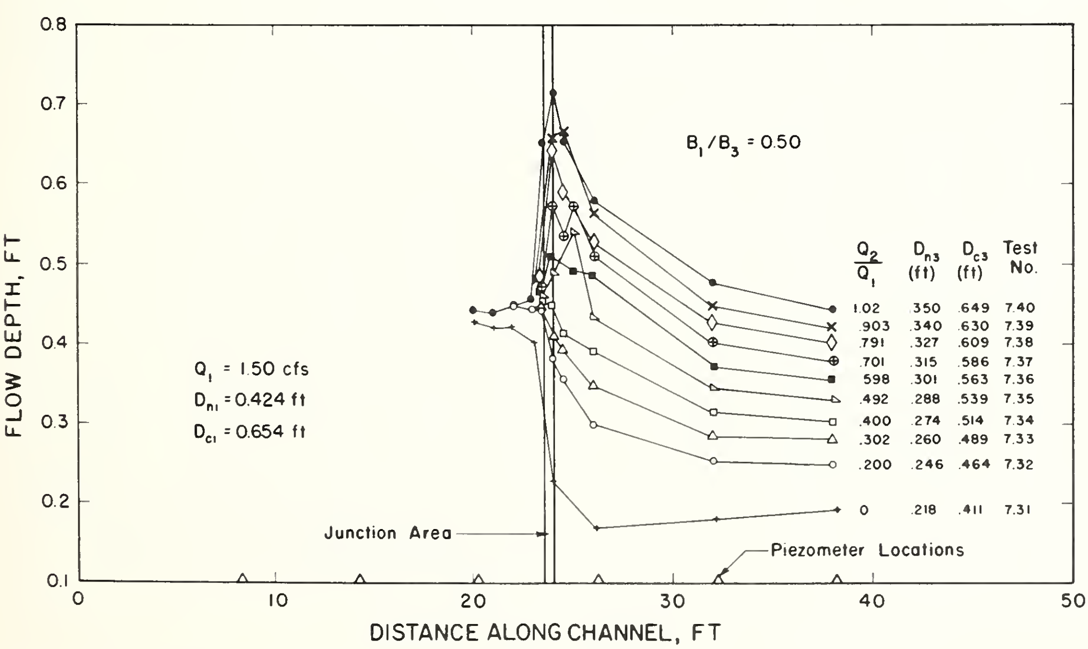


Figure 4.—Water surface profiles along main channel, test series 7, $B_1/B_2 = 0.76$, $\Theta = 30°$.

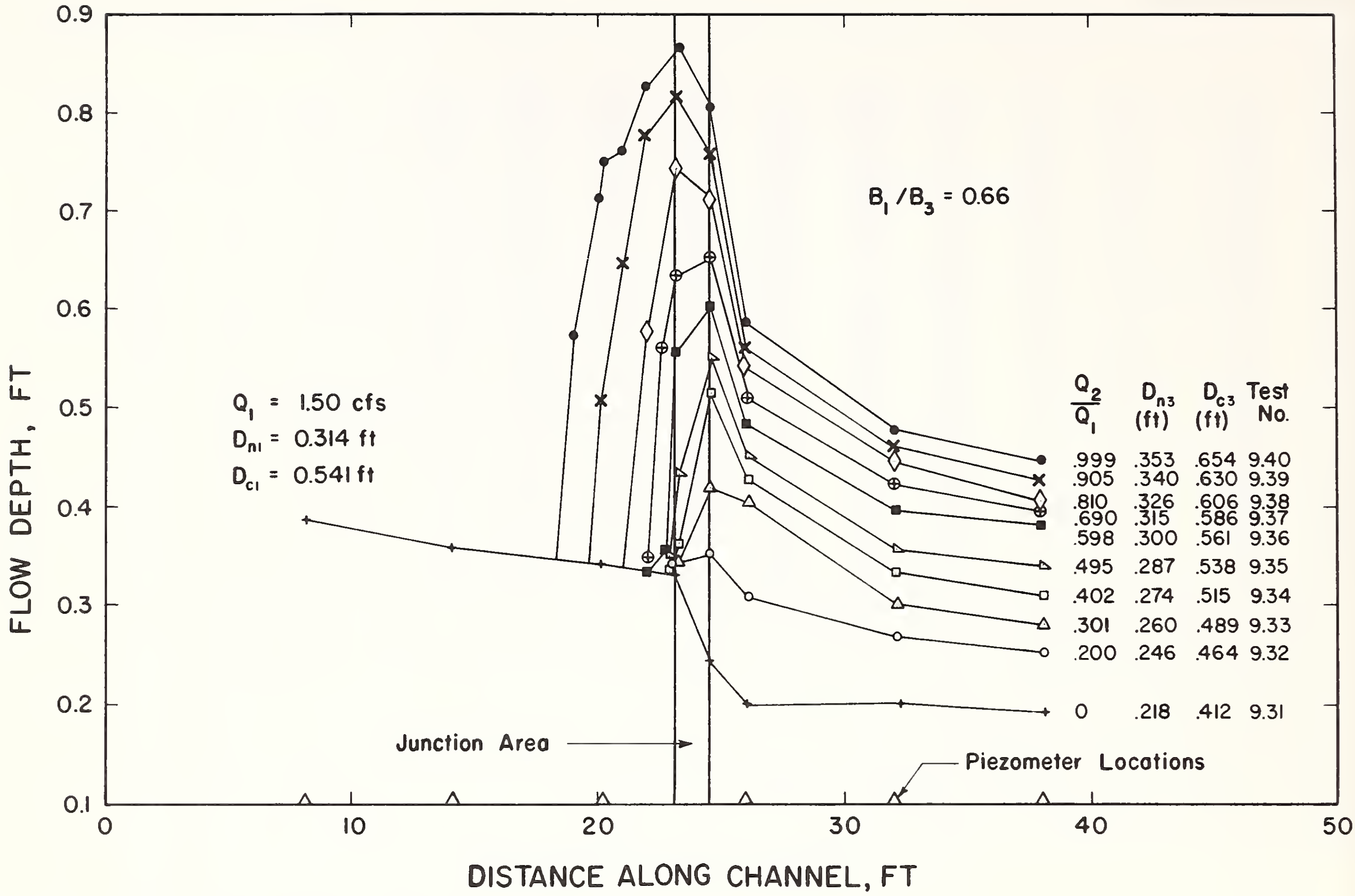


Figure 5.—Water surface profiles along main channel, test series 9, $B_1/B_2 = 0.68$, $\Theta = 30°$.

Table 5.—Series 5 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
5.01	1.498	0.339	0.199	0.338	1.697	0.339	0.365	53
5.02	1.502	.343	.300	.387	1.802	.368	.412	53
5.03	1.491	.347	.311	.394	1.802	.371	.416	53
5.04	1.501	.355	.398	.420	1.899	.403	.472	52
5.05	1.490	.359	.461	.457	1.951	.450	.497	53
5.06	1.498	.368	.503	.487	2.001	.471	.525	53
5.07	1.499	.392	.601	.524	2.100	.541	.579	53
5.08	1.490	.390	.704	.607	2.194	.584	.620	53
5.09	.749	.196	.050	.158	.799	.173	.200	54
5.10	.752	.199	.100	.202	.852	.195	.211	55
5.11	.751	.201	.150	.232	.901	.214	.253	55
5.12	.750	.203	.176	.249	.926	.225	.265	55
5.13	.755	.206	.199	.249	.954	.239	.282	54
5.14	.750	.218	.248	.276	.998	.254	.315	54
5.15	.750	.221	.300	.290	1.050	.294	.336	54
5.16	.749	.243	.349	.337	1.098	.342	.380	55

[1] Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

Table 6.—Series 6 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
6.00	0.399	0.156	0	0.017	0.399	0.077	0.156	54
6.01	.395	.158	.123	.152	.518	.155	.183	54
6.02	.398	.160	.161	.167	.559	.175	.215	55
6.03	.399	.161	.200	.168	.599	.198	.241	55
6.04	.400	.171	.239	.190	.639	.228	.270	54
6.05	.402	.179	.280	.205	.682	.262	.321	54
6.06	.396	.196	.316	.233	.712	.277	.333	54
6.07	.403	.290	.398	.292	.801	.312	.350	55
6.08	.748	.249	0	.025	.748	.124	.246	54
6.09	.751	.254	.187	.216	.938	.235	.268	55
6.10	.747	.262	.304	.275	1.051	.283	.344	55
6.11	.757	.264	.372	.307	1.129	.326	.374	56
6.12	.754	.276	.448	.328	1.202	.373	.415	56
6.13	.754	.340	.518	.352	1.272	.408	.463	56
6.14	.750	.412	.597	.397	1.347	.437	.487	56
6.15	.757	.505	.751	.464	1.508	.501	.540	55
6.17	1.202	.377	.302	.356	1.504	.349	.377	55
6.18	1.200	.386	.479	.394	1.679	.407	.455	55
6.19	1.208	.402	.595	.444	1.803	.465	.519	56
6.20	1.200	.425	.727	.498	1.927	.535	.575	56
6.21	1.203	.537	.840	.546	2.043	.580	.641	56
6.22	1.498	.422	0	.075	1.498	.228	.415	55
6.23	1.495	.447	.373	.392	1.868	.415	.451	55
6.24	1.495	.468	.599	.482	2.094	.492	.541	55
6.25	1.497	.484	.747	.532	2.244	.561	.644	55
6.26	1.501	.540	.901	.584	2.402	.630	.687	56
6.27	1.482	.636	1.053	.657	2.535	.702	.744	56
6.28	1.481	.728	1.203	.703	2.684	.743	.801	56

[1] Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

Table 7.—Series 7 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	*Tmp*
7.01	0.400	0.158	0	0	0.400	0.075	0.154	58
7.02	.402	.155	.080	.069	.482	.134	.153	58
7.03	.399	.157	.120	.062	.519	.149	.157	58
7.04	.402	.159	.160	.073	.562	.160	.158	58
7.05	.400	.158	.201	.085	.601	.160	.193	58
7.06	.398	.159	.240	.094	.638	.170	.223	59
7.07	.399	.158	.280	.107	.679	.177	.342	59
7.08	.403	.159	.303	.114	.706	.182	.315	60
7.09	.402	.157	.356	.133	.758	.196	.375	62
7.10	.392	.157	.400	.144	.792	.212	.363	62
7.50	.407	.155	.598	.202	1.005	.288	.397	64
7.11	.749	.262	0	0	.749	.134	.243	60
7.12	.750	.257	.150	.161	.900	.224	.249	60
7.13	.747	.255	.224	.179	.971	.233	.251	60
7.14	.749	.261	.304	.114	1.053	.259	.267	61
7.15	.743	.256	.375	.137	1.118	.246	.304	62
7.16	.745	.259	.450	.162	1.195	.269	.333	62
7.17	.744	.262	.521	.180	1.265	.282	.400	62
7.18	.753	.264	.601	.203	1.354	.305	.490	62
7.19	.752	.268	.675	.220	1.427	.325	.530	62
7.20	.749	.266	.751	.236	1.500	.349	.544	62
7.49	.745	.279	.903	.282	1.648	.401	.539	63
7.21	1.200	.364	0	.055	1.200	.184	.350	63
7.22	1.200	.369	.238	.273	1.438	.318	.365	61
7.23	1.204	.377	.358	.324	1.562	.354	.373	63
7.24	1.199	.380	.479	.357	1.678	.377	.403	63
7.25	1.195	.382	.602	.407	1.797	.401	.436	63
7.26	1.177	.381	.722	.237	1.899	.415	.465	63
7.27	1.211	.382	.827	.271	2.038	.424	.625	63
7.28	1.222	.395	.964	.304	2.186	.466	.731	63
7.29	1.198	.399	1.081	.334	2.279	.541	.737	63
7.30	1.197	.404	1.203	.373	2.400	.578	.789	64
7.31	1.499	.428	0	.038	1.499	.236	.406	63
7.32	1.498	.443	.299	.360	1.797	.385	.494	63
7.33	1.494	.446	.451	.396	1.945	.417	.440	63
7.34	1.496	.452	.599	.431	2.095	.455	.489	63
7.35	1.509	.461	.742	.464	2.251	.488	.632	64
7.36	1.503	.465	.899	.502	2.402	.519	.710	64
7.37	1.500	.471	1.052	.557	2.552	.577	.827	64
7.38	1.508	.480	1.193	.572	2.701	.649	.846	65
7.39	1.495	.476	1.350	.437	2.845	.666	.888	65
7.40	1.475	.658	1.498	.702	2.973	.719	.916	64
7.41	0	.131	.302	.112	.302	.119	.234	62
7.42	0	.185	.494	.172	.494	.173	.296	63
7.43	0	.232	.700	.217	.700	.219	.359	63
7.44	0	.275	.892	.277	.892	.258	.480	64
7.45	0	.313	1.100	.307	1.100	.293	.535	64
7.46	0	.360	1.305	.369	1.305	.338	.614	65
7.47	0	.402	1.502	.408	1.502	.378	.635	65

[1] Q_1 is upstream main channel discharge in cubic feet per second;
D_1 is flow depth at upstream junction station in feet;
Q_2 is lateral channel discharge in cubic feet per second;
D_2 is flow depth at lateral junction station in feet;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_m is maximum flow depth in junction vicinity in feet;
Tmp is water temperature in °F.

Table 8.—Series 8 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	Tmp
8.01	0.401	0.115	0	0.008	0.401	0.079	0.120	61
8.02	.400	.119	.080	.119	.480	.123	.145	61
8.03	.400	.117	.120	.102	.520	.138	.167	62
8.04	.398	.116	.160	.095	.558	.152	.185	62
8.05	.398	.118	.200	.086	.598	.163	.180	62
8.06	.396	.116	.245	.097	.641	.168	.196	63
8.07	.400	.120	.279	.111	.679	.180	.211	63
8.08	.397	.120	.302	.113	.699	.186	.212	63
8.09	.404	.117	.357	.129	.761	.213	.263	64
8.10	.397	.115	.400	.142	.797	.226	.287	65
8.50	.400	.375	.599	.380	.999	.363	.394	64
8.11	.753	.188	0	.025	.753	.127	.193	63
8.12	.753	.189	.149	.181	.902	.198	.220	63
8.13	.743	.191	.225	.225	.968	.222	.252	63
8.14	.747	.191	.302	.242	1.049	.255	.277	64
8.15	.749	.188	.373	.152	1.122	.284	.308	65
8.16	.748	.189	.443	.166	1.191	.270	.300	65
8.17	.749	.188	.527	.190	1.276	.297	.329	65
8.18	.745	.193	.614	.211	1.359	.346	.402	65
8.19	.753	.398	.678	.448	1.431	.443	.468	64
8.20	.751	.446	.754	.505	1.505	.477	.527	64
8.21	1.200	.273	0	.058	1.200	.187	.271	64
8.22	1.197	.281	.239	.302	1.436	.289	.319	64
8.23	1.202	.282	.359	.350	1.561	.336	.365	64
8.24	1.210	.288	.476	.399	1.686	.373	.424	64
8.25	1.201	.289	.598	.434	1.799	.437	.494	64
8.26	1.201	.295	.718	.475	1.919	.508	.585	65
8.27	1.221	.438	.828	.561	2.049	.546	.599	64
8.28	1.235	.559	.955	.618	2.190	.602	.646	65
8.29	1.203	.665	1.079	.708	2.282	.658	.720	65
8.30	1.207	.722	1.198	.740	2.405	.691	.741	64
8.31	1.499	.326	0	.069	1.499	.217	.332	65
8.32	1.496	.331	.302	.349	1.798	.348	.374	65
8.33	1.497	.339	.449	.414	1.946	.395	.448	65
8.34	1.496	.343	.598	.476	2.094	.459	.503	65
8.35	1.506	.363	.760	.526	2.266	.546	.615	65
8.36	1.486	.488	.900	.616	2.386	.610	.662	65
8.37	1.511	.609	1.042	.680	2.553	.658	.711	65
8.38	1.507	.707	1.204	.766	2.711	.726	.787	66
8.39	1.502	.790	1.341	.821	2.843	.779	.848	66
8.40	1.508	.870	1.508	.873	3.016	.838	.904	67
8.41	0	.118	.300	.113	.300	.112	.235	64
8.42	0	.200	.628	.206	.628	.192	.305	65
8.43	0	.216	.702	.226	.702	.210	.397	65
8.44	0	.261	.901	.273	.901	.253	.485	65
8.45	0	.297	1.110	.312	1.110	.292	.561	65
8.46	0	.333	1.307	.359	1.307	.316	.606	65
8.47	0	.379	1.498	.402	1.498	.359	.636	65

[1] Q_1 is upstream main channel discharge in cubic feet per second; D_1 is flow depth at upstream junction station in feet; Q_2 is lateral channel discharge in cubic feet per second; D_2 is flow depth at lateral junction station in feet; Q_3 is downstream channel discharge in cubic feet per second; D_3 is flow depth at downstream junction station in feet; D_m is maximum flow depth in junction vicinity in feet; Tmp is water temperature in °F.

Table 9.—Series 9 test data[1]

Test No.	Q_1	D_1	Q_2	D_2	Q_3	D_3	D_m	Tmp
9.01	0.400	0.120	0	0.023	0.400	0.080	0.120	64
9.02	.401	.120	.080	.114	.481	.125	.130	64
9.03	.400	.119	.120	.140	.520	.143	.160	65
9.04	.401	.121	.158	.124	.559	.161	.289	66
9.05	.403	.119	.200	.121	.603	.191	.336	66
9.06	.399	.120	.244	.169	.643	.236	.386	67
9.07	.397	.124	.280	.206	.677	.239	.389	67
9.08	.405	.126	.300	.273	.705	.247	.405	68
9.09	.396	.231	.360	.261	.756	.266	.320	68
9.10	.390	.248	.400	.293	.790	.285	.326	68
9.50	.400	.407	.599	.398	.999	.360	.421	78
9.11	.750	.190	0	.044	.750	.140	.195	67
9.12	.753	.190	.148	.190	.901	.197	.204	68
9.13	.747	.187	.224	.228	.971	.228	.280	68
9.14	.750	.193	.301	.264	1.051	.295	.487	68
9.15	.751	.191	.374	.315	1.125	.344	.396	70
9.16	.754	.196	.444	.357	1.198	.371	.414	70
9.17	.751	.306	.524	.388	1.275	.387	.444	71
9.18	.754	.375	.598	.431	1.352	.412	.458	72
9.19	.749	.434	.676	.478	1.425	.434	.496	72
9.20	.751	.485	.752	.505	1.503	.457	.534	72
9.21	1.193	.276	0	.086	1.193	.202	.399	73
9.22	1.199	.279	.236	.291	1.435	.287	.303	68
9.23	1.197	.282	.361	.333	1.558	.342	.552	73
9.24	1.203	.293	.475	.402	1.678	.433	.655	74
9.25	1.198	.298	.601	.468	1.799	.486	.705	75
9.26	1.194	.411	.721	.526	1.915	.519	.735	75
9.27	1.190	.517	.838	.577	2.028	.560	.612	76
9.28	1.209	.610	.962	.647	2.171	.606	.665	76
9.29	1.201	.690	1.083	.705	2.284	.646	.724	76
9.30	1.189	.746	1.197	.748	2.386	.678	.764	76
9.31	1.501	.331	0	.117	1.501	.242	.341	76
9.32	1.498	.337	.300	.347	1.798	.353	.365	76
9.33	1.493	.341	.450	.405	1.943	.423	.639	76
9.34	1.496	.360	.601	.495	2.097	.523	.751	76
9.35	1.500	.437	.743	.558	2.243	.564	.616	76
9.36	1.495	.568	.894	.633	2.389	.608	.657	76
9.37	1.502	.639	1.044	.697	2.546	.664	.727	76
9.38	1.481	.746	1.199	.766	2.680	.717	.787	76
9.39	1.492	.821	1.350	.832	2.842	.765	.847	76
9.40	1.502	.883	1.501	.879	3.003	.806	.901	76
9.41	0	.110	.301	.084	.301	.111	.216	68
9.42	0	.156	.502	.132	.502	.158	.303	76
9.43	0	.205	.702	.156	.702	.203	.377	76
9.44	0	.238	.897	.206	.897	.242	.438	76
9.45	0	.279	1.105	.235	1.105	.288	.510	77
9.46	0	.316	1.300	.271	1.300	.320	.578	77
9.47	0	.351	1.497	.304	1.497	.358	.634	77

[1] Q_1 is upstream main channel discharge in cubic feet per second; D_1 is flow depth at upstream junction station in feet; Q_2 is lateral channel discharge in cubic feet per second; D_2 is flow depth at lateral junction station in feet; Q_3 is downstream channel discharge in cubic feet per second; D_3 is flow depth at downstream junction station in feet; D_m is maximum flow depth in junction vicinity in feet; Tmp is water temperature in °F.

Many of the geometric and hydraulic parameters that affect the flow behavior in junctions have been mentioned in the literature review section. Others have been identified as a result of this study.

The most important parameters are the ratio of the lateral flow to the upstream main flow (Q_2/Q_1), the intersection angle (Θ), and the ratio of the upstream main to downstream main channel widths (B_1/B_3). The flow behavior is also influenced by the upstream main channel to lateral channel width ratio (B_1/B_2), and the channel slope (S). These parameters will be discussed in the following paragraphs together with such items as the effect of a drop in bed elevation as the lateral channel enters the main channel, oblique waves in the downstream channel, and the variation in depth across the channel width resulting from the flow disturbances originating at the junction.

Lateral to Upstream Main Flow Ratio

The series of photographs in figure 8 shows the effect of lateral flow on the junction flow behavior: The waves striking the far sidewall when $Q_2/Q_1 = 0$ (also shown in fig. 9*A*) are canceled as the proportion of the flow from the lateral increases; no, or minimum, waves are evident when $Q_2/Q_1 = 0.249$ and 0.401. However, a further increase in Q_2/Q_1, to 0.499, causes the lateral flow to cross the main channel creating a wave crest at the near sidewall. The height of this wave crest increases with increasing proportions of flow from the lateral (increasing Q_2/Q_1). The flow behavior with all flow from the lateral is shown in figure 9*B*. In this case, the first wave crest originates at the wall opposite the lateral channel exit.

This same information can be deduced from the water depths along and across the channels tabulated in figures 6 and 7. The data in both figures show that the

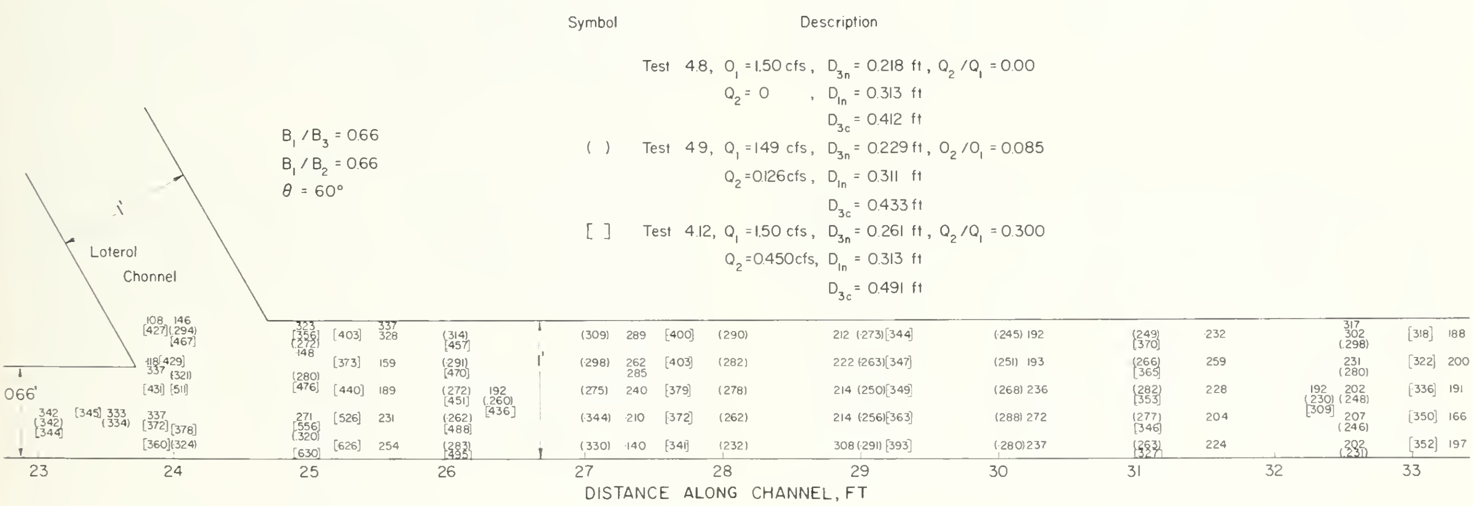


Figure 6.—Water surface depths in the main channel [ft], tests 4.8, 4.9, 4.12.

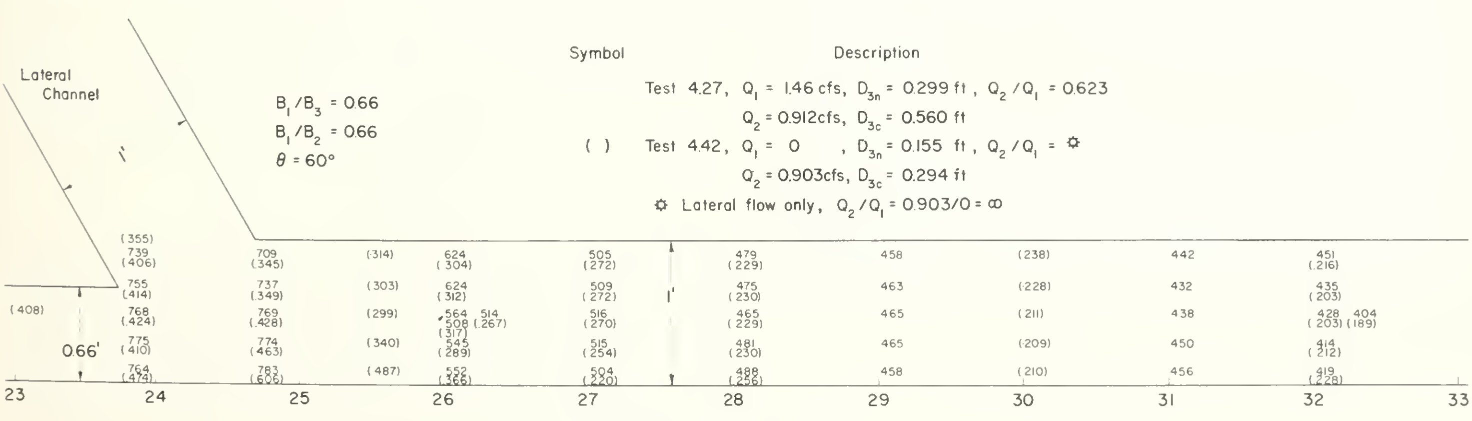


Figure 7.—Water surface depths in main channel [ft], tests 4.27, 4.42.

Figure 8.—Flow behavior in junction area, $Q_1 = 1.50$ ft^3/s, $B_1/B_2 = 0.66$, $B_1/B_3 = 0.50$: *A*, $Q_2 = 0$; *B*, $Q_2/Q_1 = 0.249$; *C*, $Q_2/Q_1 = 0.401$; *D*, $Q_2/Q_1 = 0.499$; *E*, $Q_2/Q_1 = 0.600$; *F*, $Q_2/Q_1 = 0.711$; *G*, $Q_2/Q_1 = 0.812$.

water surface elevations have more variation across the channel width with flow in one channel only. With flow in only one channel, the flow behaviors are characterized by standing oblique waves that oscillate back and forth across the downstream channel width. The first few deflections can be seen in figure 9. When both channels contribute flow, the water surface elevations across the downstream channel width are more uniform.

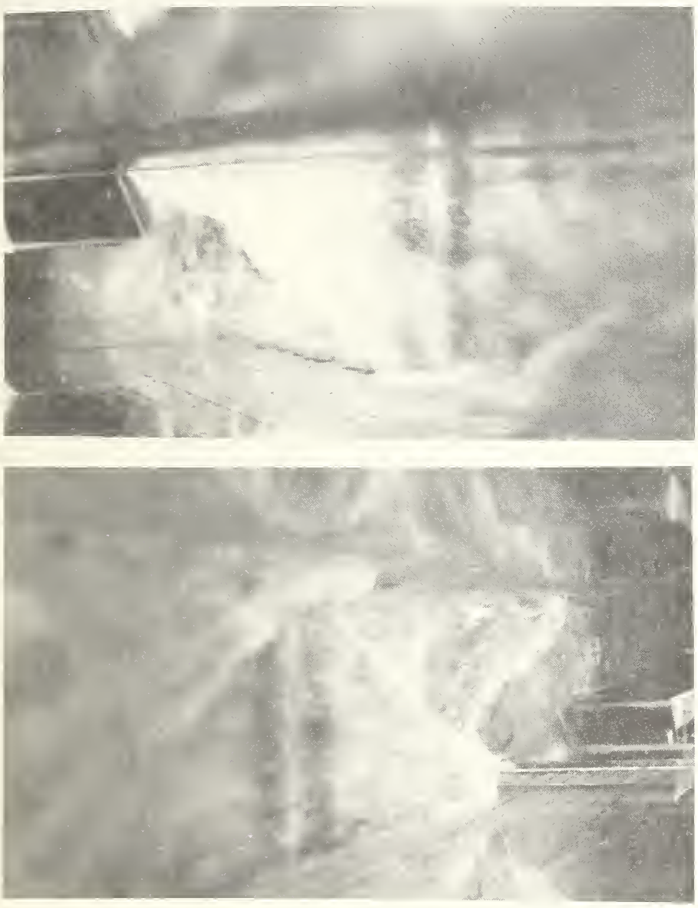


Figure 9.—Flow behavior in junction area: *A*, main channel flow only, $Q_2/Q_1 = 0$, $Q_1 = 1.50$ ft^3/s, $B_1/B_2 = 0.66$, $B_1/B_3 = 0.50$; *B*, lateral channel flow only, $Q_2/Q_1 = 1.307/0$, $B_1/B_2 = 0.67$, $B_1/B_3 = 0.67$.

Intersection Angle

Unfortunately, site conditions may make it impossible to design many open channel junctions to meet the recommendations listed in the review of literature section that minimize junction-generated standing waves and other disturbances. For example, instead of meeting the main channel at a shallow angle, the lateral may enter the main channel at angles up to and even exceeding 90°.

The effect of the intersection angle Θ on flow behavior is illustrated by comparing the data in figures 10 and 11. The intersection angles are 30° and 60°, respectively, and the channel widths and discharges are identical. The effect is shown by the increase in D_1/D_{n1} as Q_2/Q_1 increases. Since D_1 is measured at the upstream edge of the junction, any increase in D_1/D_{n1} indicates that the junction disturbance has moved into the upstream channel. Ratios Q_2/Q_1 in which D_1/D_{n1} show an increase thus indicate that the hydraulic jump has moved out of the junction area, and the research objective of containing all disturbances within the junction area has not been met. The value of Q_2/Q_1, for these two junction angles, that precludes meeting the research objective has been identified.

With $\Theta = 30°$, the jump remains in the junction area for Q_2/Q_1 values less than about 1.0 or more for all discharges shown in figure 10. With $\Theta = 60°$ (fig. 11) the jump moves upstream into the main channel at a Q_2/Q_1 value of about 0.55.

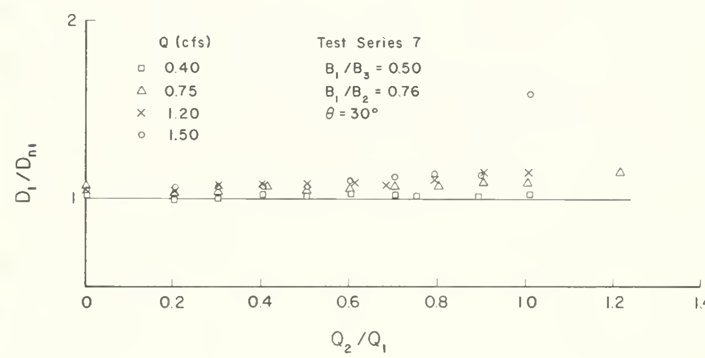


Figure 10.—D_1/D_{n1} as a function of Q_2/Q_1, test series 7.

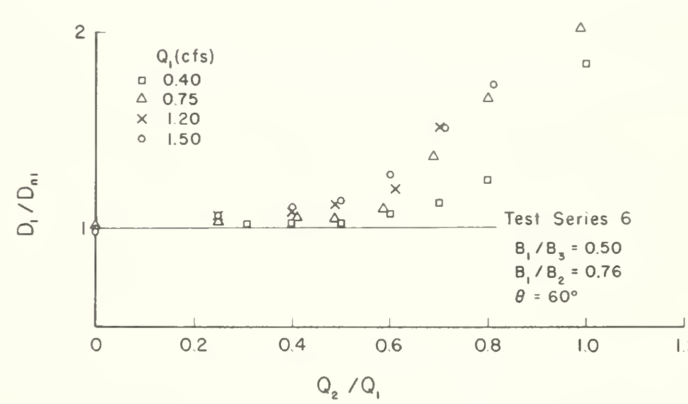


Figure 11.—D_1/D_{n1} as a function of Q_2/Q_1, test series 6.

Relative Widths of Upstream and Downstream Channels
The flow from the upstream main channel begins to spread laterally at the junction of the main channel sidewall with the upstream sidewall of the lateral. In figure 9*A*, where there is no flow from the lateral, the downstream channel is 50 percent wider than the upstream channel ($B_1/B_3 = 0.67$). The spreading flow strikes the downstream channel wall downstream of its intersection with the downstream wall of the lateral, as evidenced by the wave crest at that point.

If the upstream and downstream channels have the same width ($B_1/B_3 = 1.00$) and there is no flow from the lateral, the upstream main flow will spread into the lateral and impinge on the downstream sidewall of the lateral upstream of its junction with the downstream main. A wave will originate at that point. Fortunately, even small flows from the lateral will move the point of impingement from the lateral sidewall to the downstream main sidewall. This effect is shown in figure 8 (compare $Q_2 = 0$ with $Q_2/Q_1 = 0.249$).

The data in figures 10 and 12 illustrate the effect of B_1/B_3 on flow behavior in the junction area. The B_1/B_3 values are 0.50 and 0.67, respectively, and $\Theta = 30°$. For $B_1/B_3 = 0.50$ the jump remains in the junction area to a Q_2/Q_1 value of about 1.00 or more. For $B_1/B_3 = 0.67$ the jump remains in the junction area only to a Q_2/Q_1 value of about 0.40 for the highest Q_1; however, for the lowest Q_1 the jump did not move out of the junction area until Q_2/Q_1 exceeded about 1.10. For all tests, the jump remained in the junction area to a larger Q_2/Q_1 value for $B_1/B_3 = 0.50$ than for $B_1/B_3 = 0.67$. At $B_1/B_3 = 1.00$ only a small lateral flow ($Q_2/Q_1 \leq 0.050$) was required to cause the jump to move upstream into the main channel.

Information similar to that just discussed is shown in figures 4 and 5. For $B_1/B_3 = 0.50$ (fig. 4) the water surface elevation immediately upstream of the junction increased only for $Q_2/Q_1 = 1.02$, and the increased depth remained close to the junction. But for $B_1/B_3 = 0.67$ (fig. 5), the toe of the jump moved upstream as Q_2/Q_1 exceeded 0.40. Photographs of that effect are presented in figure 8 ($B_1/B_3 = 0.50$, $\Theta = 60°$).

The results of these tests show that the flow disturbance can be kept in the junction area—a study objective—by increasing the width of the downstream channel downstream of the junction.

Relative Widths of Upstream and Lateral Channels
The effect of the relative widths of the upstream and lateral channels (B_1/B_2) on the junction behavior is small compared with the effects of B_1/B_3 and Θ. Comparison of the data in figures 12 and 13 for flows and channel widths that are identical except for the lateral channel shows that the jump moves upstream of the junction at lesser values of Q_2/Q_1 for the wider lateral channel than for the narrower lateral. The results thus seem to indicate that, based on the study objective criteria, the relatively narrower lateral channel improves the junction flow behavior.

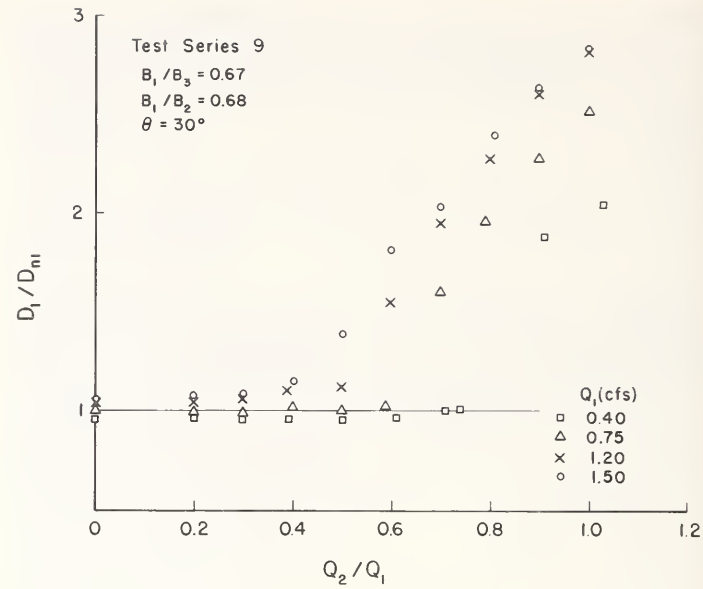


Figure 12.—D_1/D_{n1} as a function of Q_2/Q_1, test series 9.

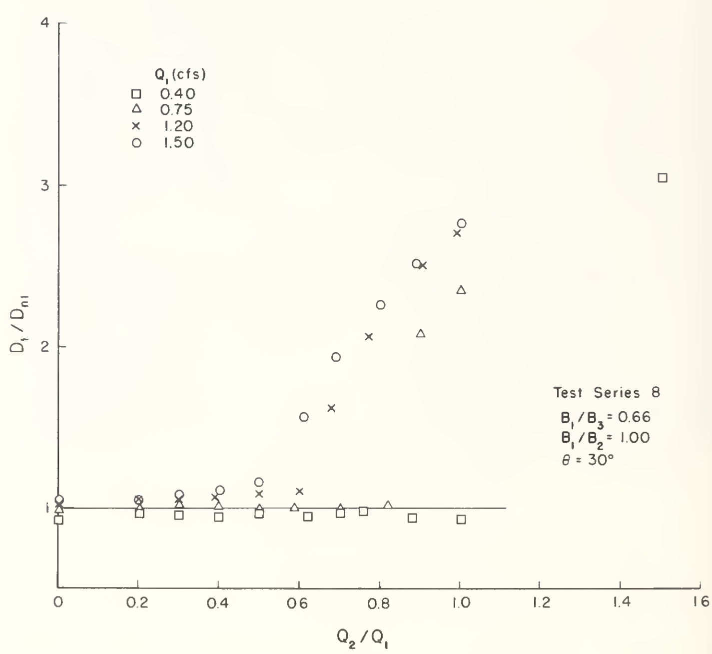


Figure 13.—D_1/D_{n1} as a function of Q_2/Q_1, test series 8.

Effect of Channel Slope

Despite the meager data available for determining the effect of channel slope, the data in figures 14 and 15 seem to indicate that the junction performance is improved by increasing the slope of the main channel. These data are for 90°-intersection angle junctions and main channel slopes of 0.00465 and 0.0198, respectively. The jump moves upstream at a Q_2/Q_1 value of about 0.1 for the flatter slope compared with a Q_2/Q_1 value of about 0.3 for the steeper slope. This indicates that the junction performance is improved by increasing the slope of the main channel.

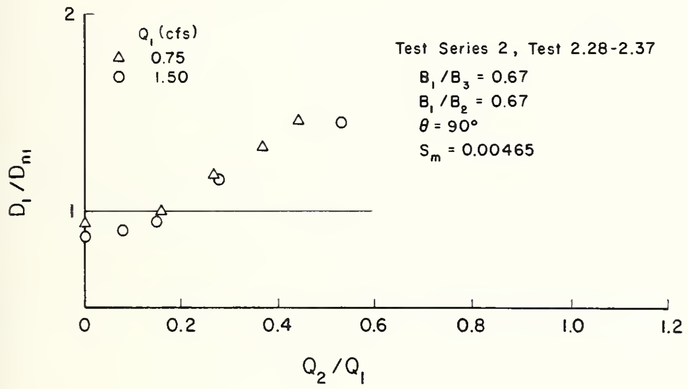


Figure 14.—D_1/D_{n1} as a function of Q_2/Q_1, test series 2.

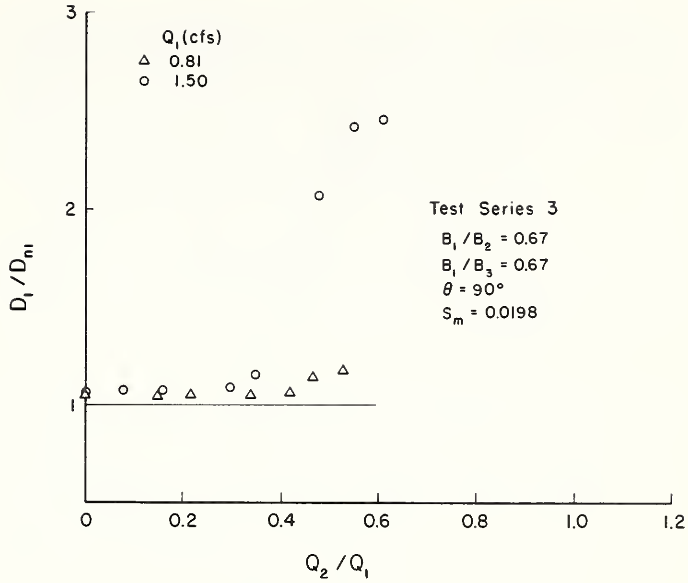


Figure 15.—D_1/D_{n1} as a function of Q_2/Q_1, test series 3.

Bed Elevation Difference

A few tests with the lateral channel bottom at a higher elevation than the main channel bottom showed that bed elevation did not significantly affect flow behavior in the junction area.

Depth of Flow

The major problem in the design of open channel junctions with supercritical flow is to determine the water surface elevations in the channel junction and vicinity. These elevations are then used to calculate the required heights of the channel sidewalls. Important to the accurate calculation of the sidewall heights are the maximum depth in the junction area D_m, the depth at the lateral junction exit D_2, the depth at the entrance to the downstream channel D_3, and the heights of the oblique waves caused by junction disturbances in the downstream channel.

Maximum Depth in the Junction Area

The maximum depth in the junction area D_m usually occurs along the sidewall opposite the lateral channel exit. The maximum depths for the tests are plotted in figure 16 as a function of the depth at the lateral channel entrance D_2 or D_m/D_2 and the Froude number F_2 for the flow condition at the lateral channel exit. The maximum values of D_m/D_2 occur with low upstream main channel flows and with no upstream main channel flow.

There is considerable scatter of the data with no recognizable trend that will permit development of a general relationship to predict D_m/D_2 as a function of the Froude number F_2. A D_m/D_2 value of 3.2 envelopes all the D_m/D_2 values obtained, but use of this value for design would result in significant overdesign of the sidewall heights for many of the variable combinations.

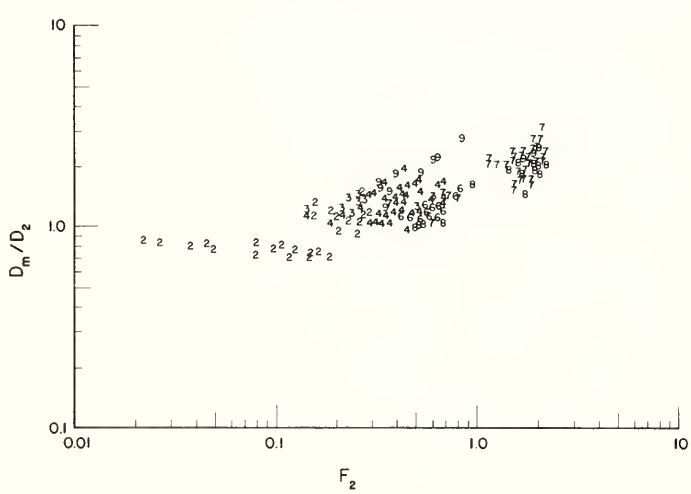


Figure 16.—D_1/D_{n1} as a function of F_2.

The ratio of the maximum depth D_m to the depth sequent to the normal depth in the upstream main channel D_{squ} is plotted in figure 17 as a function of Q_2/Q_1. For some of the variable combinations the maximum depth in the junction area exceeded the sequent depth of the upstream channel D_{squ}. The data are scattered with no apparent trend to permit the development of a general prediction equation for D_m.

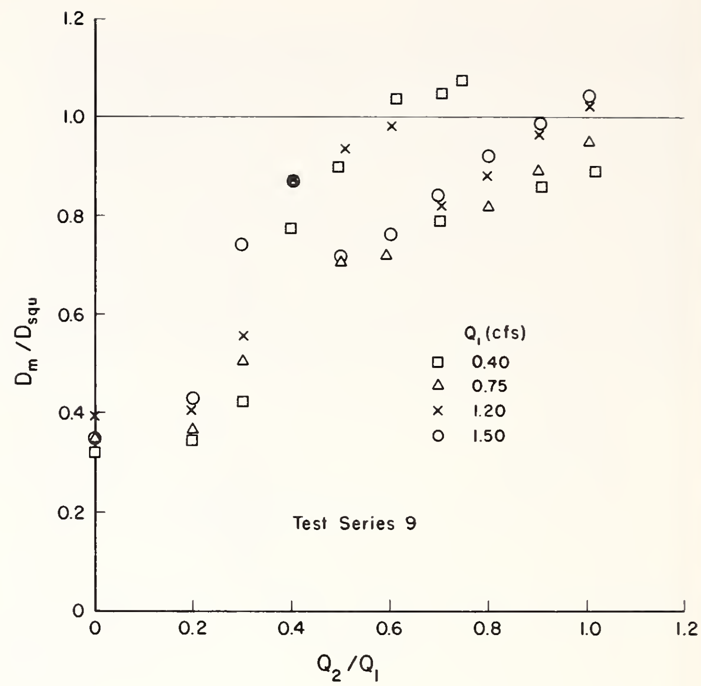


Figure 17.—D_m/D_{squ} as a function of Q_2/Q_1, test series 9, $B_1/B_3 = 0.67$, $B_1/B_2 = 0.98$, $\Theta = 30°$.

Depth at Lateral Channel Exit

The depth D_2 at the lateral channel exit must be known to calculate the lateral channel sidewall heights. A general relationship to predict D_2 could not be developed from the observed results. However, the maximum designed depth in the junction area adjacent to the lateral channel will always be greater than the lateral channel exit wall height, so the junction sidewall height can be used as the lateral channel exit sidewall height.

The D_m/D_2 (maximum D_2/D_m) values were minimum under the conditions of test series 2: $\Theta = 90°$, $S_m = 0.00465$, $B_1/B_3 = 1.00$, and $B_1/B_2 = 1.00$. For this junction widths are equal for upstream and downstream mains, and the upstream main flow can expand into the lateral and impinge on the lateral downstream wall, especially when lateral flow is absent and lateral elevation is equal to that of the main. In all other test series values of both B_1/B_3 and Θ were lower than those in series 2, and flow conditions were better and increases in water surface elevation were smaller. The maximum value of D_2/D_m (1.02) was the only value greater than 1.0.

Depth at Entrance to Downstream Channel
The water surface elevations downstream of the junction must be known to calculate the downstream channel sidewall heights. Typical water surface profiles are presented for test series 7 and one value of Q_1 in figure 4 and for test series 9 and two values of Q_1 in figures 5 and 18. For most of the variable combinations tested, normal depth in the downstream channel D_{n3} was not achieved at the last piezometer location 13.5 ft downstream from the junction. Figure 18 provides a direct comparison between observed and normal

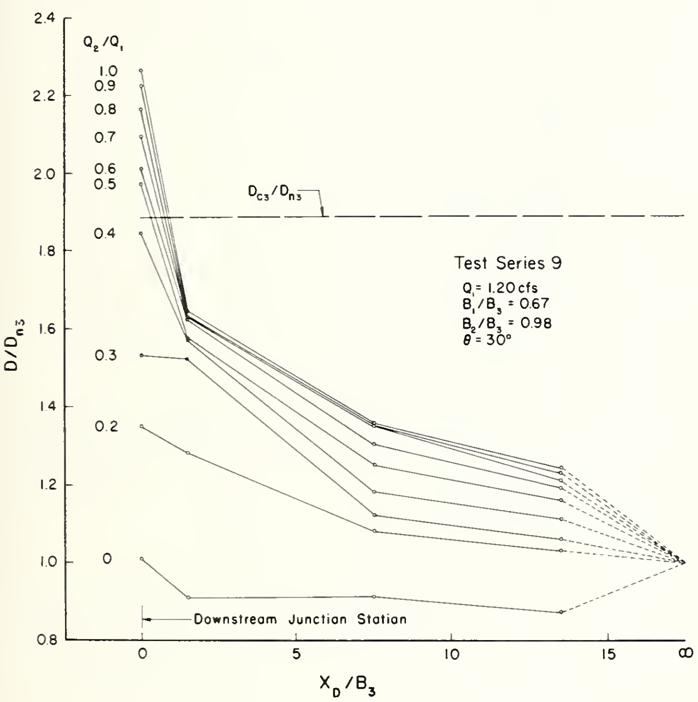


Figure 18.—Normalized water surface profiles along main channel, test series 9.

depths, and the D_{c3}/D_{n3} line provides a comparison between the observed and critical depths in the downstream channel. As Q_2/Q_1 increases, observed depth increases more than does the normal depth, and greater distance is required before observed depth approaches normal depth.

The profiles downstream of the entrance to the downstream channel can be adequately predicted, with common methods of calculation, by assuming that the depth at the station downstream from the junction equals the critical depth. Since the flow downstream of the junction is supercritical, the profile must be calculated by starting with D_{c3} at the downstream channel entrance (the junction exit) and computing in a downstream direction.

For some tests, observed and calculated flow depths 13.5 ft downstream of the junction are presented in table 10. The profiles were calculated by the direct step method, with both D_{c3} and D_3 as initial flow depths. Values of D_{co}/D_o and D_{cc}/D_o, which are defined in table 10 footnotes, are presented.

Use of the observed depth D_3, instead of critical depth D_{c3}, as the initial flow depth at the downstream channel entrance improves agreement with the observed depths 13.5 ft downstream by an average of 3 percent. Use of the different depth assumptions at the junction exit, however, produces only small differences. Calculated depths are larger than observed depths. The difference might be attributed, in part, to the use of a constant Manning coefficient for the calculation of profiles. Overall, the magnitudes of the differences in downstream water surface profiles calculated with observed depth D_3 and with critical depth D_{c3} are so small that use of the more easily determined D_{c3} seems justified.

Oblique Wave Heights in Downstream Channel
The disturbance in flow behavior caused by the junction may create a standing oblique wave pattern in the channel downstream of the junction. The oblique wave pattern is more pronounced when there is upstream main channel flow only or lateral channel flow only. The data obtained during these preliminary tests gave only indications of the effects of oblique waves on the downstream channel flow depths. Some of these effects will be mentioned.

The data in figure 6 for test 4.8 show that, with no flow from the lateral, the depths at station 25 vary from 0.148 to 0.323 ft, 2.2 times the minimum. The high read ings along the left wall downstream of the junction result from the wave deflected from that wall by the spreading flow from the upstream main. Even at station 30, 5 ft downstream from the junction, the depth variation across the channel is from 0.192 to 0.272 ft. For test 4.9 there is a small lateral flow, $Q_2/Q_1 = 0.085$, and most of the oblique wave pattern is suppressed. For test 4.12 the lateral to upstream main flow ratio Q_2/Q_1 is 0.30, and the oblique waves are suppressed except for a distance of about one ft downstream from the junction.

The data in figure 7 for test 4.42 are for lateral flow only. The high depths along the right wall just downstream of the junction indicate the height of the wave caused by flow from the lateral striking the wall opposite the junction. For test 4.27, with flow from the upstream main, $Q_2/Q_1 = 0.623$, the wave is suppressed, and there is only about a 20 percent variation in depth across the width of the channel for about 2 ft downstream from the junction. Farther downstream the oblique wave pattern is almost completely suppressed.

Table 10.—Observed and calculated depths of flow in downstream channel[1]

Test No.	Q_1	Q_3	D_3	D_c	D_{n3}	D_{cc}/D_{n3}	D_o	D_{cc}	D_{cc}/D_o	D_{co}	D_{co}/D_o
3.03	1.49	1.618	0.278	0.433	0.230	1.21	0.223	0.278	1.25	0.253	1.13
3.09	1.50	1.741	.321	.455	.241	1.22	.247	.295	1.19	.277	1.12
3.10	1.48	1.922	.481	.486	.259	1.23	.281	.318	1.13	.318	1.13
4.48	.399	.449	.105	.184	.099	1.06	.095	.105	1.11	.099	1.04
4.49	.398	.498	.131	.197	.106	1.19	.095	.113	1.19	.110	1.16
4.50	.398	.059	.213	.223	.119	1.05	.123	.129	1.05	.129	1.05
4.66	1.201	1.400	.273	.393	.208	1.19	.215	.248	1.15	.234	1.09
4.67	1.207	1.505	.302	.412	.218	1.20	.236	.261	1.11	.251	1.06
4.68	1.208	1.707	.414	.443	.238	1.21	.273	.289	1.06	.288	1.05
6.09	.751	.938	.235	.301	.159	1.14	.161	.182	1.13	.178	1.11
6.10	.747	1.051	.283	.324	.172	1.15	.179	.198	1.11	.197	1.10
6.11	.757	1.129	.326	.340	.180	1.17	.194	.210	1.08	.209	1.08
6.23	1.495	1.868	.415	.476	.253	1.23	.253	.310	1.23	.307	1.21
6.24	1.495	2.094	.492	.514	.274	1.24	.301	.339	1.13	.338	1.12
7.34	1.496	2.095	.455	.514	.274	1.23	.306	.337	1.10	.334	1.09
7.36	1.503	2.402	.519	.563	.301	1.25	.356	.375	1.05	.375	1.05
7.37	1.500	2.552	.577	.586	.315	1.25	.378	.393	1.04	.393	1.04
8.02	.400	.480	.123	.193	.104	1.05	.102	.109	1.07	.106	1.04
8.05	.398	.598	.163	.223	.119	1.08	.126	.129	1.02	.126	1.00
8.08	.397	.699	.186	.247	.132	1.09	.140	.144	1.03	.142	1.01
8.13	.743	.968	.222	.307	.163	1.14	.163	.185	1.13	.179	1.10
8.15	.749	1.122	.284	.339	.179	1.16	.197	.207	1.05	.205	1.04
8.18	.745	1.359	.346	.385	.204	1.18	.236	.240	1.02	.239	1.01
8.22	1.197	1.436	.289	.400	.212	1.18	.220	.251	1.14	.240	1.09
8.24	1.210	1.686	.373	.445	.236	1.20	.264	.284	1.08	.280	1.06
8.25	1.201	1.799	.437	.465	.247	1.21	.285	.299	1.05	.298	1.05
8.32	1.496	1.798	.348	.464	.246	1.22	.260	.299	1.15	.287	1.10
8.33	1.497	1.946	.395	.490	.261	1.22	.289	.318	1.10	.310	1.07
	Averages					1.16			1.10		1.07

[1] Q_1 is upstream main channel discharge in cubic feet per second;
Q_3 is downstream channel discharge in cubic feet per second;
D_3 is flow depth at downstream junction station in feet;
D_{n3} is normal flow depth in downstream channel in feet;
D_{cc} is calculated flow depth in feet about 13.5 ft downstream of junction assuming $D_3 = D_{c3}$;
D_{c3} is critical flow depth in downstream channel in feet;
D_o is observed flow depth in feet about 13.5 ft downstream of junction;
D_{co} is calculated flow depth in feet about 13.5 ft downstream of junction using D_3 for depth at downstream channel entrance;
A Manning coefficient of 0.0086 was used to calculate normal flow depths and water surface profiles.

The data in figure 19 for test series 3 are for a junction with an intersection angle of 90° which causes, with lateral flow, the most pronounced oblique wave pattern and highest wave crest elevations of any angle tested. The solid lines are the centerline profiles and the plotted points are the maximum wave crest heights observed in the main channel. Wave crest heights were maximum along the wall opposite the lateral at the point where the wave was deflected from the wall.

Assuming that depth of flow at the downstream channel entrance equals critical depth, computed depths of flow in the downstream channel are averages. For some variable combinations sidewalls higher than average may be required to compensate for the oblique wave crest heights.

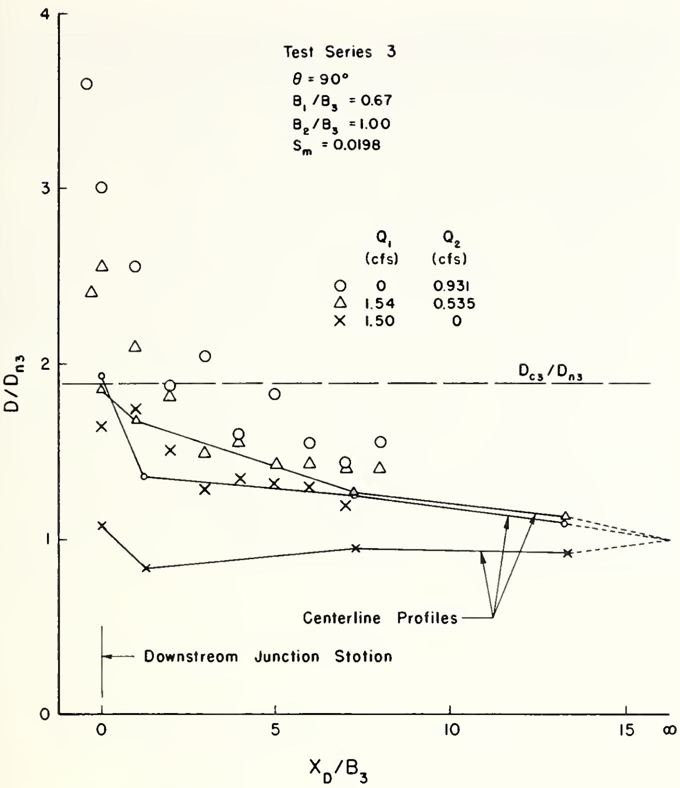


Figure 19.—D/D_{n3} as a function of distance below downstream junction station, test series 3.

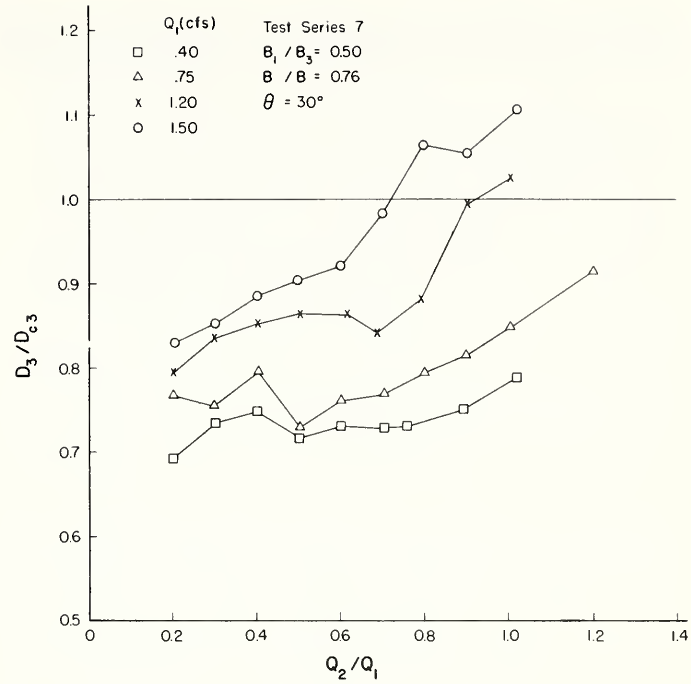


Figure 20.—D_3/D_{c3} as a function of Q_2/Q_1, test series 7.

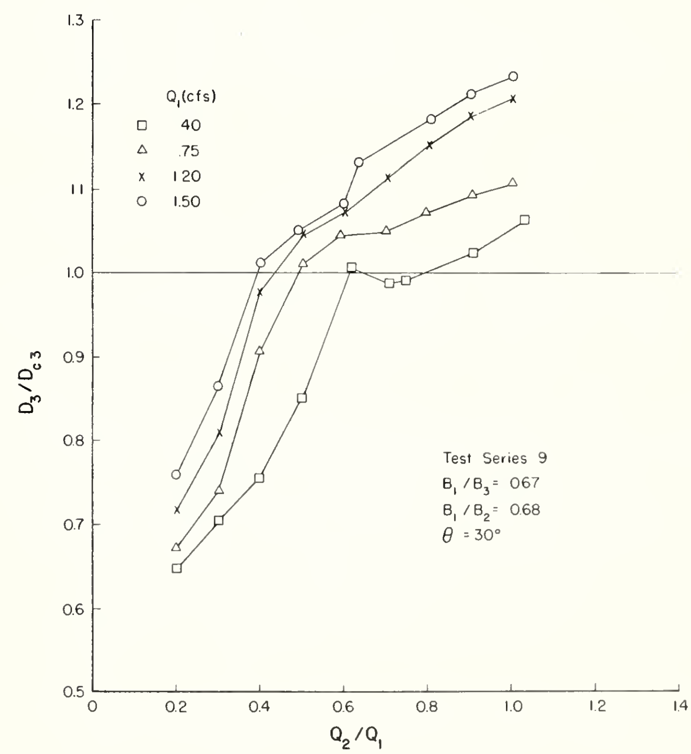


Figure 21.—D_3/D_{c3} as a function of Q_2/Q_1, test series 9.

Flow Conditions in Junction Area

For calculating the water surface profile in the downstream channel, results are satisfactory when critical depth is assumed equal to depth measured at the entrance to the downstream channel; that is, $D_3 = D_{c3}$. The same assumption is made for calculating flow conditions within the junction: satisfactory values for the depth at the downstream junction station can be calculated by assuming that D_3 is equal to critical depth D_{c3}. Although this assumption is not valid for all flow conditions, the approximation is adequate for junction design. The ratio D_3/D_{c3} (observed depth/critical depth at downstream station) as a function of Q_2/Q_1 is presented in figures 20 and 21 for two test series. The D_3 values vary from less than critical depth (D_3/D_{c3} less than 1.0) at small Q_2/Q_1 values to greater than critical depth at large Q_2/Q_1 values. Observations during the tests show that the Q_2/Q_1 value at which $D_3/D_{c3} = 1.0$ is slightly less than the Q_2/Q_1 value at which the jump begins to move upstream into the main channel. When D_3/D_{c3} becomes equal to 1.0, flow becomes subcritical within the junction, and additional increases in lateral flow cause the jump to move upstream—contrary to the study objectives.

The design objective is to maintain the jump within the junction area and not permit it to move upstream of the junction into the upstream main channel. As long as the downstream station depth D_3 is less than critical depth D_{c3}, no jump will form in the junction. If D_3 is equal to or greater than D_{c3}, a jump will form in the junction. The jump can be contained within the junction area by limiting the lateral inflow Q_2 so that Q_2/Q_1 is less than some maximum value. The depth in the main channel upstream of the junction then can be computed as the normal depth. If Q_2/Q_1 is greater than the maximum value, the jump will move upstream into the main channel. For this situation, the distance the jump moves upstream and the maximum depth in the junction area cannot be calculated or predicted with confidence.

Analysis and study of the observed test data demonstrated that the momentum relationship, based on some simplifying assumptions, could be used to adequately predict the variable values required to contain the jump, if one forms, within the junction area. The momentum relationship, equation 2, was modified to more nearly represent the observed data. Equation 2 was developed for tranquil flow conditions, and assumes that the water surface elevations are equal at the upstream main channel and lateral channels. However, the data showed that, for conditions contributing to the formation of a jump in the junction, water surface elevation at the lateral channel exit station was usually more nearly equal to the water surface elevation at the downstream than at the upstream junction station. Equation 2, modified to consider that relation, becomes:

$$\frac{Q_3^2}{gB_3D_3} + \frac{B_3D_3^2}{2}$$

$$= \frac{Q_1^2}{gB_1D_1} + \frac{B_1D_1^2}{2} + \frac{Q_2^2}{gB_2D_3}(\cos \Theta) + \left(\frac{B_3 - B_1}{2}\right)D_3^2 \quad \ldots\ldots(3)$$

The junction dimensions (B_1, B_2, B_3, and Θ); test flows (Q_1, Q_2, and Q_3); and observed depths (D_1 and D_3) were used in computing the upstream momentum flux M_u (sometimes called specific force) at the junction entrance—the right side of equation 3—and the downstream momentum flux M_d at the junction exit—the left side of equation 3. The ratio M_u/M_d is plotted in figure 22 for several junction parameter combinations.

If each parameter in equation 3 were known exactly, the sides of the equation would be equal—upstream momentum flux M_u would equal downstream momentum flux M_d. However, not all parameters in equation 3 are known exactly, so the equation does not balance.

For small Q_2/Q_1 values flux M_u was greater than flux M_d. As Q_2/Q_1 increased, a value was reached at which calculated upstream and downstream momentum flux were equal ($M_u = M_d$). When Q_2/Q_1 exceeded that value, flux M_u was less than flux M_d ($M_u < M_d$). When flux M_u and flux M_d were equal, average depth at the downstream junction station was near critical depth which is at, or near, the Q_2/Q_1 value at which the jump starts to move upstream of the junction into the upstream main channel. That value of Q_2/Q_1 cannot be exceeded if the design objective of containing the jump within the junction area is to be met. An example will show how the downstream channel width can be calculated to ensure that the jump will remain within the junction area. The results in figure 22 indicate that the upstream main to downstream channel width ratio B_1/B_3 has a much greater influence on the flow behavior in the junction area than does the intersection angle Θ.

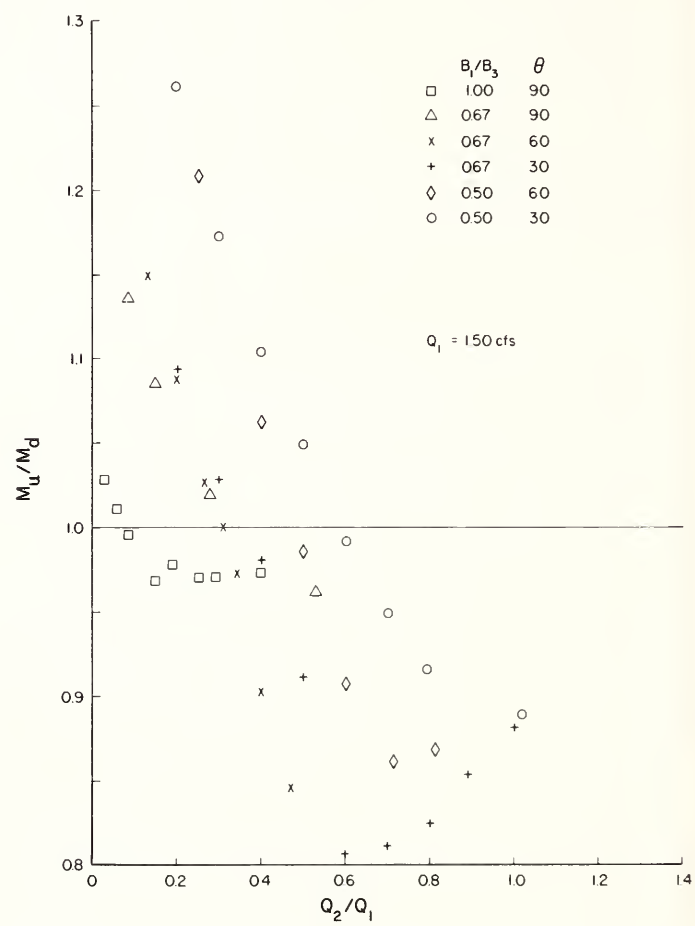


Figure 22.—M_u/M_d as a function of Q_2/Q_1.

Downstream Channel Width

By trial and error, equation 3 can be used to determine the downstream channel width B_3 at which the upstream and downstream momentum fluxes are equal ($M_u = M_d$ and $M_u/M_d = 1.0$).

In equation 3, the parameters B_1, B_2, and Θ are known from the junction geometry. Hydrologic computations determine Q_1, Q_2, and $Q_3 = Q_1 + Q_2$. The upstream normal depth is computed by use of the upstream channel parameters. The downstream depth D_3 is the critical depth calculated from Q_3 and B_3.

The calculations are performed by assuming a value of B_3 and calculating the upstream and downstream momentum flux. If the upstream to downstream momentum flux ratio M_u/M_d is not one, B_3 should be appropriately adjusted and the momentum flux calculated again. Values of B_3 should be tested until the momentum flux ratio is 1.0.

An example illustrates the procedure.

Given: $S_l = S_m = 0.02$ $\quad Q_1 = 150$ ft³/s
$n = 0.013$ $\quad Q_2 = 60$ ft³/s
$B_1 = 6.0$ ft $\quad Q_2/Q_1 = 0.40$
$B_2 = 4.0$ ft $\quad \Theta = 60$ degrees

The successive computations are presented in the following tabulation:

B_3 (ft)	D_1* (ft)	D_3** (ft)	M_u (ft^3)	M_d (ft^3)	M_u/M_d
6.0	1.528	3.365	87.48	101.9	0.858
6.5	1.528	3.190	90.26	99.20	.910
7.0	1.528	3.036	92.54	96.78	.956
7.5	1.528	2.900	94.46	94.58	.999
8.0	1.528	2.777	96.08	92.57	1.04

* D_1 is normal depth in the upstream main channel.
** D_3 is critical depth in the downstream channel.

In these calculations, the first assumption was $B_3 = B_1$. Since M_u/M_d is less than one, the B_3 width is not sufficient. So, B_3 is increased in ½-ft increments until, with $B_3 = 7.5$ ft, $M_u/M_d = 0.999$. Thus, 7.5 ft is the minimum width downstream channel that can be used. Use of a width greater than 7.5 ft, say 8.0 ft, would provide a factor of safety.

Calculation of channel width sizes the horizontal dimensions of the channel junction. Vertical dimensions, wall heights, must be sized by use of appropriate data for maximum depths in the junction area and the calculated downstream water surface profile. The following paragraphs summarize the procedures for evaluating the variables involved in the design of open channel junctions where the flow is supercritical.

Width of Downstream Channel

The downstream channel width is calculated by trial and error by use of equation 3 as demonstrated in the previous example and illustrated with the successive computations in the above tabulation. The minimum downstream channel width is that which causes the calculated upstream momentum flux M_u to equal the calculated downstream momentum flux M_d ($M_u = M_d$ and $M_u/M_d = 1.0$). This minimum width should be increased by 5 percent or so to ensure satisfactory flow performance in the junction area by containing the jump, if one forms, in the junction area.

Maximum Depth in Junction Area

The downstream channel width is calculated so that the flow disturbance created in the junction by the intersecting upstream main and lateral channel flows will not cause a hydraulic jump to form in the junction and move upstream into the upstream main channel. At the maximum design discharge values (Q_1, Q_2, and Q_3), this flow disturbance in the junction will have a depth less than the depth sequent to the normal depth in the upstream main channel.

Table 11 presents variable and parameter values and data for tests in which the ratios of the calculated downstream momentum flux M_d to the upstream momentum flux M_u are nearest to, but not greater than, 1.0. The maximum observed depths D_m from table 11 are plotted versus the depths sequent to the normal depth in the upstream main channel D_{squ} in figure 23. For these tests, which satisfy the design objective of containing the jump within the junction area, the maximum observed depth D_m never equaled or exceeded the sequent depth in the upstream main channel. The maximum depth D_m to sequent depth D_{squ} ratios (D_m/D_{squ}) varied from 0.487 to 0.939 for these tests. Only two ratios exceeded 0.90, and most were between 0.70 and 0.90.

A relationship to predict the maximum depth D_m in the junction area as a function of the geometric and hydraulic parameters could not be developed. However, the depth sequent to the normal depth in the upstream main can be used as the design depth for the channel sidewall height in the junction area. The observed depths indicate that use of the sequent depth for the junction sidewall height will be conservative for many parameter combinations but provides a factor of safety for all the parameter combinations tested. Although the design sidewall heights will be conservative for many situations, the length of channel wall involved is not great, and the cost for the additional sidewall height will not be excessive.

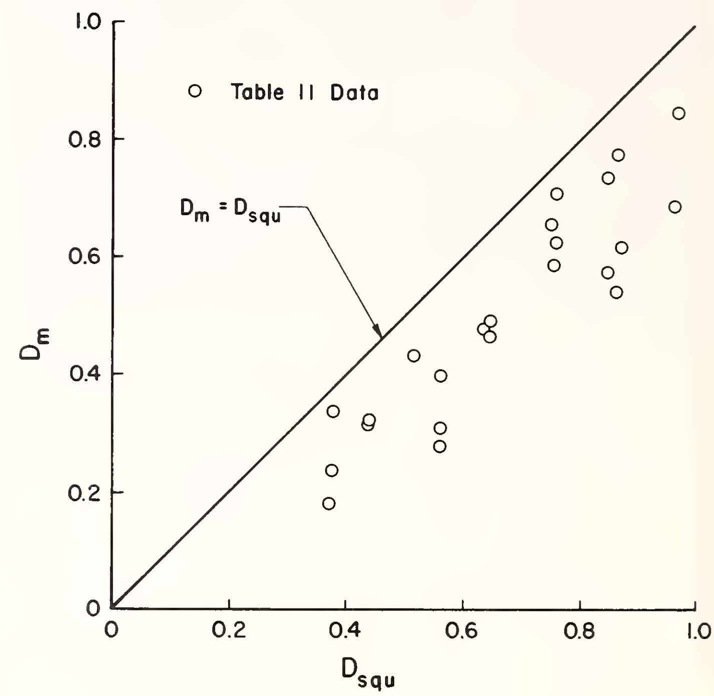


Figure 23.—D_m as a function of D_{squ}.

Depth at Upstream Channel Exit

The downstream channel width is calculated so that the flow disturbance in the junction area created by the intersecting upstream main and lateral channel flows will not cause a hydraulic jump to form and move upstream into the upstream main. Thus, the depth in the upstream main can be calculated as the normal depth, which can be used for designing the channel sidewall height in the upstream main.

The sidewall height at the upstream main channel exit will be the same as the sidewall height in the junction, equal to the depth sequent to the normal depth in the upstream main channel. So, a transition from the upstream main sidewall height to the upstream main exit sidewall height should be made by sloping the top of the upstream main sidewall to intersect the top of the upstream main exit sidewall. This transition should extend a distance upstream of the junction equal to, or greater than, about 10 times the upstream main channel width.

Table 11.—Variable and parameter values and observed data for tests with $M_u/M_d > 1.00$[1]

Test No.	B_1	Q_1	B_2	Q_2	Q_2/Q_1	Q_3	D_{c3}	D_m	D_{squ}	D_m/D_{squ}	M_u/M_d
4.56	0.67	0.657	0.66	0.296	0.421	0.953	0.304	0.421	0.512	0.822	1.073
4.62	.67	.910	.66	.417	.523	1.40	.393	.475	.630	.829	1.008
4.68	.67	1.21	.66	.499	.623	1.71	.448	.623	.754	.827	1.080
4.75	.67	1.51	.66	.695	.775	2.20	.532	.775	.864	.897	1.037
5.08	.67	1.49	.66	.704	.472	2.19	.530	.620	.858	.748	1.031
5.16	.67	.749	.66	.349	.466	1.10	.334	.380	.557	.709	1.059
6.05	.50	.402	.66	.280	.697	.682	.243	.321	.434	.740	1.043
6.13	.50	.754	.66	.518	.687	1.27	.368	.463	.641	.722	1.025
6.20	.50	1.20	.66	.727	.606	1.93	.486	.575	.844	.681	1.040
6.26	.50	1.50	.66	.901	.600	2.40	.563	.687	.959	.716	1.028
7.08	.50	.403	.66	.303	.752	.706	.249	.315	.435	.728	1.086
7.18	.50	.753	.66	.601	.798	1.35	.384	.490	.641	.765	1.051
7.29	.50	1.20	.66	1.08	.902	2.28	.544	.737	.843	.874	1.004
7.38	.50	1.51	.66	1.19	.791	2.70	.609	.846	.962	.880	1.018
8.05	.67	.398	.67	.200	.503	.598	.223	.180	.369	.487	1.085
8.15	.67	.749	.67	.373	.498	1.12	.339	.308	.558	.552	1.079
8.26	.67	1.20	.67	.718	.598	1.92	.485	.585	.751	.779	1.017
8.35	.67	1.51	.67	.760	.505	2.27	.542	.615	.864	.712	1.050
9.05	.67	.403	.98	.200	.496	.603	.234	.336	.372	.902	1.058
9.15	.67	.751	.98	.374	.498	1.12	.340	.396	.559	.708	1.048
9.25	.67	1.20	.98	.601	.502	1.80	.465	.705	.751	.939	1.032
9.35	.67	1.50	.98	.743	.495	2.24	.538	.616	.862	.714	1.024

[1] B_1 is upstream main channel width in feet;
Q_1 is upstream main channel discharge in cubic feet per second;
B_2 is lateral channel width in feet;
Q_2 is lateral channel discharge in cubic feet per second;
Q_3 is downstream channel discharge in cubic feet per second;
D_{c3} is critical flow depth in downstream channel in feet;
D_m is maximum flow depth in junction vicinity in feet;
D_{squ} is flow depth sequent to normal flow depth in upstream main channel in feet;
M_u is upstream momentum flux in cubic feet;
M_d is downstream momentum flux in cubic feet.

Depth at Downstream Channel Entrance

For the design condition in which the upstream momentum flux M_u is equal to or greater than the downstream momentum flux M_d ($M_u/M_d \geq 1.0$), the test results indicate that the flow depths at the junction downstream station are about equal to the critical depth in the downstream channel D_{c3}. However, the sidewall height at the downstream channel entrance will be equal to the sidewall height in the junction; that is, the sequent depth in the upstream main channel.

Depth in the Downstream Channel

The depth profiles downstream of the entrance to the downstream channel can be adequately predicted, with common methods of calculation, by assuming that the depth at the station downstream from the junction equals the critical depth D_{c3}. Except for a short distance immediately downstream of the junction, the calculated depths are slightly larger than the observed depths. So, the calculated profiles should provide adequate design values for the average downstream channel sidewall height.

With no flow from the lateral and with supercritical flow in the upstream main, or with supercritical flow in the lateral channel and no flow in the upstream main, a wave is created that oscillates back and forth across the downstream channel until it is gradually dissipated by friction. It was assumed previously that containment of the crest heights of these oscillating waves would require elevated sidewalls for a considerable distance downstream of the junction. The data, mostly from test 7.38, that are presented in figure 24, however, indicate that the assumption is not necessarily valid. For test 7.38, $M_u/M_d = 1.018$ and satisfies the design condition that $M_u/M_d \geq 1.0$. Maximum observed depths D_m are pre-

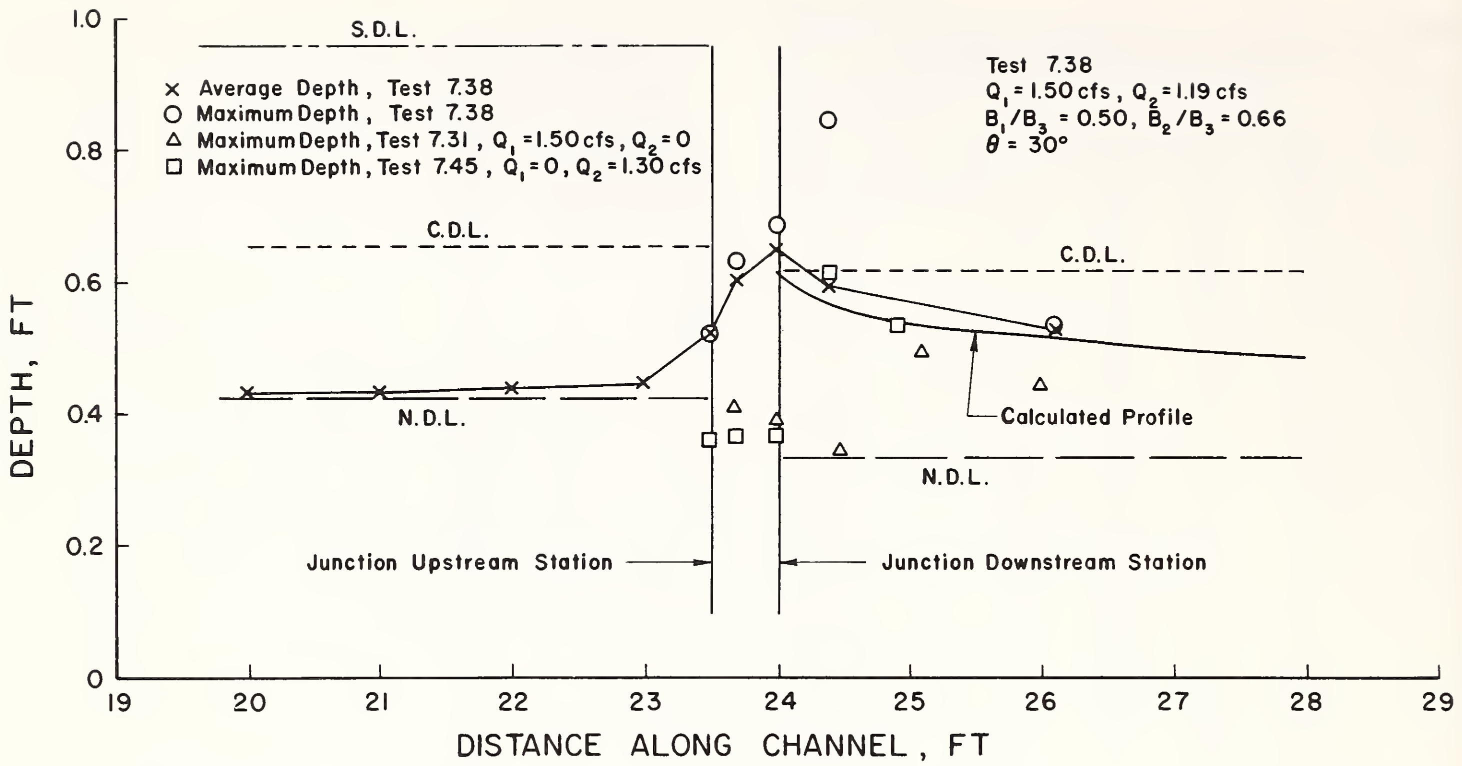


Figure 24.—Water surface profile along main channel, test 7.38.

sented with $Q_2 = 0$(test 7.31) and with $Q_1 = 0$(test 7.45) for comparison.

The maximum depth was highest ($D_m = 0.846$ ft) with the design condition in test 7.38, not with either lateral channel flow only ($D_m = 0.614$ ft) in test 7.45 or upstream main channel flow only ($D_m = 0.490$ ft) in test 7.31. These results are confirmed by the data in figures 6 and 7; depths in the downstream channel are greater with combined flows than with either lateral channel flow only or upstream main channel flow only. Also, the chance is very small that either maximum-design lateral channel flow only or maximum-design upstream main channel flow only would occur and probably should not be a significant consideration in the design of the downstream channel sidewalls.

The test results indicated that the superelevations sometimes extended into the channel downstream of the junction for distances up to about 5 ft ($X_d/B_3 = 5.0$). Thus, the height of the downstream channel sidewalls must be elevated for a short distance downstream of the downstream junction station. This can be accomplished by sloping the top of the downstream channel sidewall to intersect with the top of the junction sidewall at the station downstream from the junction. This sloping transition should have a length equal to about 10 times the downstream channel width ($X_d/B_3 = 10$).

Depth at Lateral Channel Exit

For the design condition in which the upstream momentum flux M_u is equal to or greater than the downstream momentum flux M_d ($M_u/M_d \geq 1.0$), the ratio (D_m/D_2) of the maximum depth D_m to the lateral channel exit depth D_2 varied from 1.10 to 3.18. The depth in the lateral channel exit exceeded the maximum depth in the junction area only once ($D_2/D_m = 1.02$). Thus, for design, the channel sidewall height at the lateral channel exit will be the same as the junction sidewall height, equal to the sequent depth in the upstream main channel.

Depth in Lateral Channel

A hydraulic jump may or may not form at the lateral channel exit and move upstream into the lateral channel. Predicting when a jump will form and move upstream into the lateral channel is not possible with the test results obtained. For design, the assumption must be made that a jump will form and move upstream into the lateral channel.

The lateral channel sidewall height can be calculated with the procedures presented by Chow (1959,

pp. 398-403) to determine the location and length of a hydraulic jump. For calculating the lateral channel sidewall height, a horizontal line is extended upstream from the top of the sidewall at the lateral channel exit until it intersects the depth line sequent to the normal flow in the lateral channel plus a distance equal to the calculated length of the hydraulic jump (Chow, fig. 15, p. 398) in the lateral channel. The line can then be extended horizontally until it intersects the normal depth line for the lateral channel. The elevation difference between this line and the channel bottom is equal to the channel sidewall height. Upstream of the jump the depth in the lateral channel will be equal to the normal depth, which can be used for the upstream lateral channel sidewall height.

Suitable freeboard heights should be added to each depth, remembering that the required freeboard for supercritical flow greatly exceeds that normally used for subcritical flows. However, the sidewall heights calculated from the depth sequent to the normal depth in the upstream channel already have a freeboard added to them by virtue of the safety factor implicit in the approach.

If correctly applied, the approach described for tests with the model should work for full-sized structures. The procedure should be appropriate, and probably conservative, for channel slopes that are steeper than those tested. The jump should stay within the junction area to larger Q_2/Q_1 values for the steeper slopes compared with the slopes tested. However, because the steeper sloped channels would have larger Froude numbers than do the channels tested, the maximum elevations in the junction area might be larger than those observed in these tests.

The flow behavior in the vicinity of the confluence of two open channels with steep slopes is affected by complex combinations of many variables and the development of general criteria for the design of junctions is difficult. The momentum principle, developed for tranquil flow conditions and applied to the supercritical flows, did not accurately predict the flow depths in the junction area. Modifications based on these tests should give satisfactory values for junction design.

For the range of conditions studied the most important variables affecting the flow behavior in the junction area were the upstream main channel flow-lateral channel flow combination, the upstream main channel to downstream channel width ratio, and the intersection angle between the main and lateral channels. The slope effect was not adequately evaluated.

The results demonstrated that the downstream channel width should be greater than the upstream main channel width for acceptable flow behavior in the junction area. For a given flow combination, the flow behavior in the junction area improves as the upstream main to downstream channel width ratio B_1/B_3 and/or the intersection angle Θ becomes smaller. For a given junction geometry, the flow behavior in the junction area becomes more turbulent and flow depths greater as the lateral channel to upstream main channel discharge ratio Q_2/Q_1 increases except when $Q_2 = 0$. With lateral channel or upstream main channel flow only, an oblique wave pattern occurs in the downstream channel below the junction.

The channel junction should be designed so that the jump created by the intersecting lateral channel and

upstream main channel flows will remain within the junction area and not move into the upstream main channel. The downstream channel width required to contain the jump within the junction area can be calculated, by trial and error, with equation 3. The design conditions to contain the jump within the junction area will be met if the upstream momentum flux M_u (right side of eq. 3) is equal to, or greater than, the downstream momentum flux M_d (left side of eq. 3) ($M_u/M_d \geq 1.0$). This design condition ensures that depth remains normal in the upstream main channel and that the downstream channel flow depth remains equal to or less than critical depth. A jump may form at the lateral channel exit and move upstream into the lateral channel.

The maximum depth D_m usually occurs along the sidewall opposite the lateral channel entrance. The location of this maximum depth D_m is a function of the intersection angle Θ and may be within the junction or a short distance downstream of the junction. For the tests that satisfy the design condition of containing the jump within the junction area, the maximum depth varied from about 0.49 to about 0.94 times the depth sequent to the normal depth in the upstream main channel.

The flow depth at the lateral channel exit (D_2) varied from about 0.31 to about 0.91 times the maximum depth in the junction (D_m).

The depth at the entrance to the downstream channel (D_3) can be approximated by the critical depth in the downstream channel (D_{c3}). Since all normal flow depths in the downstream channel were considerably less than critical depth, the flow in the downstream channel was always supercritical. The downstream channel water depths can be adequately predicted by assuming that depth at the entrance to the downstream channel equals critical depth and calculating the supercritical flow water surface profile in the downstream channel by any of the common methods.

The results of this study, if used with discretion, provide valuable guidance for the design of open channel junctions with supercritical flow. The use of appropriate channel geometry should achieve the objective and ensure that the jump created by the intersecting lateral channel and upstream main channel flows be contained within the junction area. For a given flow condition, the channel widths and/or the intersection angle can be selected to meet that objective. The design principles developed for moderate slopes should be adaptable for steeper slopes. As the slope increases, the jump should remain within the junction area when Q_2/Q_1 values are greater than those for the slopes tested.

Tests under various specific conditions should help in measuring the effects of different variables on flow behavior in the junction area and in developing general design criteria so that open channel junctions with supercritical flow can be adequately, confidently, and economically designed. Also, verification of the data from the model in tests with prototype structures would provide confidence in the design procedure.

Literature Cited

Behlke, C. E., and Pritchett, H. D.
1966. Design of supercritical flow control junctions. Highway Research Record No. 123. Highway Research Board, National Research Council, Washington, D.C.

Bowers, C. E.
1950. Hydraulic model studies for Whiting Field naval air station. Part V; Studies of Open-Channel Junctions. Saint Anthony Falls Hydraulic Laboratory Project Report No. 24.

Chow, Ven Te.
1959. Open-channel hydraulics. 680 p. McGraw-Hill Book Co.

Department of the Army, Corps of Engineers, Office of the Chief of Engineers.
1970. Hydraulic Design of Flood Control Channels. EM 1110-2-1601, 67 pp., 6 appendixes.

Gildea, A. P., and Wong, R. F.
1967. Flood control channel hydraulics. Proceedings, 12th Congress, International Association for Hydraulic Research 1:330-337.

Greated, C. A.
1968. Supercritical flow through a junction. LaHouille Blanche 8:693-695.

Taylor, Edward H.
1944. Flow characteristics at rectangular open-channel junctions. Transactions of the American Society of Civil Engineers 109:893-903.

Webber, N. B., and Greated, C. A.
1966. An investigation of flow behavior at the junction of rectangular junctions. Proceedings, Institute of Civil Engineers 34:321-334.

Wong, R. F., and Robles, A., Jr.
1971. Flood-control facilities for unique flood problems. Proceedings, American Society of Civil Engineers, Journal of Waterways, Harbors, and Coastal Engineering Division, Volume 97, No. WW1, February, pp. 185-203.

*U. S. Government Printing Office: 1985 0 - 525-674 (30173)

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